BRIDGE 28W – 112TH AVE SE OVER SB 1-405

I-405; RENTON TO BELLEVUE WIDENING AND EXPRESS TOLL LANES PROJECT

NDC57 RFC Submittal
Design Calculations

October 25, 2021

Designer:	Rashim Garg
Checker:	Mehdi Dastfan
Supervisor:	Matt Baughman

PROJECT: **WSDOT I-405 R2B - BR 28W**

CLIENT: **Wood PLC**

CONT: **A207833**

Book: E6

Title: Piles

CALC BOOK COMPLETION DATE: **2021 Sep 15**

Item	Page #'s	to	Page #'s	Subject / Description	Designer's Initials	Checker's Initials	Comments
1	E6-0	-	E6-0	Calc Register (this sheet)	N/A	N/A	
2	E6-1	-	E6-1	Piles Analysis Assumptions/Methodology	RSGR	MEDN	
3	E6-2	-	E6-6	Piles Axial Load Estimation	RSGR	MEDN	
4	E6-7	-	E6-9	Piles GA and depth estimation	RSGR	MEDN	
4	E6-10	-	E6-15	Piles Lateral Load estimation	RSGR	MEDN	
5	E6-16	-	E6-16	Load Cases	RSGR	MEDN	
6	E6-17	-	E6-25	L Pile Analysis and Structural Checks	RSGR	MEDN	
7	E6-26	-	E6-27	Structural Check for Jacking Load Case	RSGR	MEDN	
8	E6-28	-	E6-35	Flexure Capacity of Composite Section	RSGR	MEDN	
9	E6-36	-	E6-37	Pile Top Flexure Capacity	RSGR	MEDN	
10	E6-38		E6-42	Drawings	RSGR	MEDN	
11	E6-43		E6-53	Geotech Addendum	RSGR	MEDN	

Design Summary

Assumptions:

1. Downdrag has been considered from superstructure, pile cap and Fin Walls and are added to downdrag experienced at pile tip. See Page 06 for downdrag profile. Downdrag on pile in contact with soil is ignored in the CMP sleeve region.

2. No soil imposed deformations are considered on pile due to CMP sleeve.

3. Axial loads from superstructure are considered. Lateral loads include loads from bearing deformation (friction), earthquake, lateral earth pressure and wind. Cases with, without and partial bearing deformation are considered. When including the bearing deformation (FR), if the deflection at top is analyzed to be 1.5"+ (at which bearing force will act opposite to other lateral forces) then bearing deformation is removed or reduced.

The governing cases for both abutments shown on page 16

4. Axial load bearing capacities are provided by the geotechnical engineer. See page 08 and 09.

5. Total transverse seismic load in abutment 1 is reduced to account for resistance offered by 1 Fin walls. The calculations for transverse load reduction on piles due to fin wall is on page 10.

6. Total transverse seismic load in abutment 2 is reduced to account for resistance offered by passive pressure on curtain wall. 85% passive pressure is assumed as the curtain wall is expected to move the required value to mobilize full passive pressure. The calculations for reductions are on page 11 and 85% mobilization assumption is verified on page 22.

7. The demand/capacity for piles at abutment 1 (West) in EXT case is about 90% and in STR case is about 66%. The majority of demand is due to flexure in EXT case.

8. The demand/capacity for piles at abutment 2 (East) in EXT case is about 62% and in STR case is about 62%.

9. Capacity is based on Reinforced concrete filled steel tube composite section as per WSDOT BDM section 7.10. See page 28 to 35. The concrete and reinforcement are terminated 30' below pile head based on the moment diagram.

Load Estimation

NOTES

Unfactored Dead Load

Abutment	DC [kips]	DW [kips]
1	1399	97
2	1420	96

from PTWL Superstructure Dead Loads + weight of substructure

Nos. of piles

Abutment	Nos.
1	7
2	6

Load per pile:

Abutment	DC [kips]	DW [kips]
1	213	14
2	250	16

Total Load/nos. of piles

*DC Load includes weight of superstructure + pile cap + curtain walls + Fin Wall + Soil above Fin Wall + approach slab + concrete in piles
**DW is load from wearing surface

Unfactored Live Load (Max. in a pile)

Design Lanes

Roadway Width

Abutment 1 length	51.2	ft
Abutment 2 length	54.8	ft
Deisgn Traffic Lanes	2.0	lanes
Possible Lanes at Abutment 1	4.0	lanes
Possible lanes at Pier 2/Abutment 2	4.0	lanes

Base configuration

MPF - Substructure

Number of Loaded Lanes	m
1	1.20
2	1.00
3	0.85
>3	0.65

Multiple Presence Factor
Reduced MPFs for substructures per BDM 3.9.3 C / AASHTO 3.6.1.1.2

Dynamic Allowance IM (DLA)

Substructure 0.33

AASHTO Table 3.6.2.1-1

Design Live Load HL-93

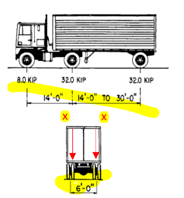
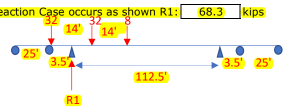


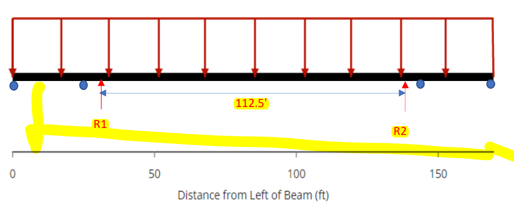
Figure 3.6.1.2.2-1—Characteristics of the Design Truck

Max Reaction Case occurs as shown R1: 68.3 kips

=((32(112.5+3.5))+(32*(112.5+3.5-14)+(32*(112.5-14+3.5-14)))/(112.5*



Lane Load 0.64 k/ft



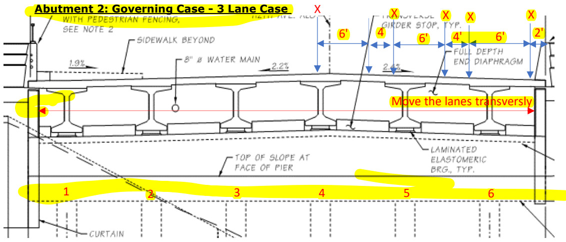
Max Reaction due to lane load : 46 kips

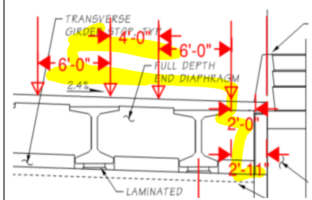
In order to calculate the value of X, the following cases were checked:

		2X (kips)		X (kips)
Case 1	1 Design Truck	68	kips	34
Case 2	(1 Design Truck + Lane Load)	114	kips	57
Case 3	(1 Design Truck_DLA + Lane Load).	137		68

Use Case 2 and 3 to get maximum reaction on piles due different lane arrangement (inc. on sidewalk), see below

Abutment 2: Governing Case - 3 Lane Case





2 Lane Case

Abutment width, w=	54.75	ft	use abutment 2 for estimate of live load for both abutments
Distance of piles centre from edge, c=	27.50	ft	
Total Load for 2 lanes, 4x=	229	kips	(includes MPF)
No. of Piles, n=	6	nos.	
Distance of centre of 2 lanes from edge, L2=	11	ft	Lane placed at about 2'-11" from edge of abutment
Moment if 2 lanes moved to abut. centre, M=	3790	k-ft	
Distance of pile 6 from the abut centre, S6=	24	ft	
Distance of pile 5 from the abut centre, S5=	14	ft	
Distance of pile 4 from the abut centre, S4=	4	ft	
Distance of pile 3 from the abut centre, S3=	-5	ft	
Distance of pile 2 from the abut centre, S2=	-15	ft	
Distance of pile 1 from the abut centre, S1=	-24	ft	
ΣS^2	1614		
Max Load in Pile 6=	94	kips	$=P/n + (M*S)/\Sigma S^2$
Max Load in Pile 5=	71	kips	
Max Load in Pile 4=	47	kips	
Max Load in Pile 3=	26	kips	
Max Load in Pile 2=	3	kips	
Max Load in Pile 1=	-18	kips	

3 Lane Case

Abutment width, w=	54.75	ft	
Distance of abut centre from edge, c=	27.50	ft	
Total Load for 3 lanes, 6x=	291	kips	(includes MPF)
No. of Piles, n=	6	nos.	
Distance of centre of 3 lanes from edge, L3=	16	ft	Lane placed at about 2'-11" from edge of abutment
Moment if 3 lanes moved to abut. centre, M=	3375	k-ft	
Distance of pile 6 from the abut centre, S6=	24	ft	
Distance of pile 5 from the abut centre, S5=	14	ft	
Distance of pile 4 from the abut centre, S4=	4	ft	
Distance of pile 3 from the abut centre, S3=	-5	ft	
Distance of pile 2 from the abut centre, S2=	-15	ft	
Distance of pile 1 from the abut centre, S1=	-24	ft	
ΣS^2	1614		
Max Load in Pile 6=	99	kips	$=P/n + (M*S)/\Sigma S^2$
Max Load in Pile 5=	78	kips	
Max Load in Pile 4=	57	kips	
Max Load in Pile 3=	38	kips	
Max Load in Pile 2=	17	kips	
Max Load in Pile 1=	-2	kips	

This figure shows 3 lane example, which is placed at different locations along the pile cap to get max reaction
Similarly is checked for 1, 2 and 4 lanes, but 3 lane case is the maximum

Max Reaction occurs at Pile 6 due to 3 lanes	99	kips
Max Reaction occurs at Pile 6 due to 3 lanes (w/ DLA)	118	kips

COWI

PROJECT:	I-405 R2B Final Design - BR 28W (112th Ave)			JOB NO.:	A207833
SUBJECT:	Pile Loadings and Analysis - Load Estimates			Sheet:	1
BY:	RSGR			DATE:	9/13/2021
CHECK:	MEDN			DATE:	9/13/2021

Abutment 1: Governing Case - 3 Lane Case

2 Lane Case

Abutment width, w=	51.21	ft	use abutment 2 for estimate of live load for both abutments
Distance of piles centre from edge, c=	25.60	ft	
Total Load for 2 lanes, 4x=	229	kips	(includes MPF)
No. of Piles, n=	7	nos.	
Distance of centre of 2 lanes from edge, L2=	11	ft	Lane placed at about 2'-11" from edge of abutment
Moment if 2 lanes moved to abut. centre, M=	3357	k-ft	
Distance of pile 7 from the abut centre, S7=	19.63	ft	
Distance of pile 6 from the abut centre, S6=	13.08	ft	
Distance of pile 5 from the abut centre, S5=	6.54	ft	
Distance of pile 4 from the abut centre, S4=	0	ft	
Distance of pile 3 from the abut centre, S3=	-7	ft	
Distance of pile 2 from the abut centre, S2=	-13	ft	
Distance of pile 1 from the abut centre, S1=	-20	ft	
ΣS^2	1198		
Max Load in Pile 7=	88	kips	
Max Load in Pile 6=	69	kips	$=P/n + (M*S)/\Sigma S^2$
Max Load in Pile 5=	51	kips	
Max Load in Pile 4=	33	kips	
Max Load in Pile 3=	14	kips	
Max Load in Pile 2=	-4	kips	
Max Load in Pile 1=	-22	kips	

3 Lane Case

Abutment width, w=	51.21	ft	
Distance of abut centre from edge, c=	25.60	ft	
Total Load for 3 lanes, 6x=	291	kips	(includes MPF)
No. of Piles, n=	7	nos.	
Distance of centre of 3 lanes from edge, L3=	16	ft	Lane placed at about 2'-11" from edge of abutment
Moment if 3 lanes moved to abut. centre, M=	2823	k-ft	
Distance of pile 7 from the abut centre, S7=	20	ft	
Distance of pile 6 from the abut centre, S6=	13	ft	
Distance of pile 5 from the abut centre, S5=	7	ft	
Distance of pile 4 from the abut centre, S4=	0	ft	
Distance of pile 3 from the abut centre, S3=	-7	ft	
Distance of pile 2 from the abut centre, S2=	-13	ft	
Distance of pile 1 from the abut centre, S1=	-20	ft	
ΣS^2	1198		
Max Load in Pile 7=	88	kips	
Max Load in Pile 6=	72	kips	$=P/n + (M*S)/\Sigma S^2$
Max Load in Pile 5=	57	kips	
Max Load in Pile 4=	42	kips	
Max Load in Pile 3=	26	kips	
Max Load in Pile 2=	11	kips	
Max Load in Pile 1=	-5	kips	

This figure shows 3 lane example, which is placed at different locations along the pile cap to get max reaction
Similarly is checked for 1, 2 and 4 lanes

Max Reaction occurs at Pile 7 due to 3 lanes	88	kips
Max Reaction occurs at Pile 7 due to 3 lanes (w/ DLA)	105	kips

COWI

PROJECT:	I-405 R2B Final Design - BR 28W (112th Ave)	JOB NO.:	A207833
SUBJECT:	Pile Loadings and Analysis - Load Estimates	Sheet:	1
BY:	RSGR	DATE:	9/13/2021
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LOAD FACTORS (AASHTO Ta. 3.4.1-1)

	DEAD		LL
	DC	DW	
Strength I	1.25	1.50	1.75
Strength V	1.25	1.50	1.35
Extreme I	1.00	1.00	0.50

FACTORED DL AND LL LOADS (per pile)

Abutment 1 (West)

	αDEAD	αLL	αLL_DLA	Total	Total_DLA
Strength I	287	154	184	441	471
Strength V	287	119	142	405	429
Extreme I	227	44	-	271	-

Abutment 2 (East)

	αDEAD	αLL	αLL_DLA	Total	Total_DLA
Strength I	336	173	207	509	543
Strength V	336	133	160	469	496
Extreme I	266	44	-	310	-

FACTORED DOWNDRAG LOADS - Downdrag Profile

1.0 DD From Superstructure, Pile cap

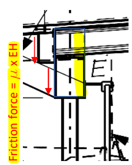
1.1 From end diaphragm and pile cap

height of pile cap, hpc= 6.5 ft
height of diaphragm, hs= 7.6 ft
Total height of soil, h= 11.7 ft

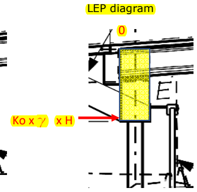
Length of abutment, L1= 51.21 ft
Length of abutment, L2= 54.75 ft

Soil unit weight γ= 135 pcf
LEP coefficient, static, Ko= 0.384

Pile cap bottom level below ground, H= 11.7 ft
Lateral EP at bottom of pile cap= 0.60653 k/ft2
LEP Force at bottom of pile cap abut 1, EH1= 182 kips
LEP Force at bottom of pile cap abut 2, EH2= 194 kips



LEP diagram



1.2 Calculate Friction Force between concrete and soil
Downdrag is equal to friction developed

Φ'F= 25.46 =2/3 of 38 degrees friction angle (for concrete and soil)
Friction coefficient, μ= TAN δ=TAN Φ'F= 0.48

			=divide by pile nos.	Factored (x 1.4)
Friction Force along pile cap abut FR1=	87	=μ x EH1	12 kips per pile	17 kips per pile
Friction Force along pile cap abut FR2=	92	=μ x EH2	15 kips per pile	22 kips per pile

2.0 DD From Fin Walls and Soil above Fin Walls (Only occurs in Abutment 1)

2.1 From Fin Wall (Concrete-soil)

Value given by geotech
Total Friction Force due to 1 Fin Wall= 31.0 kips — =divide by pile nos. 4 kips per pile — Factored (x 1.4) 6

2.1 From Soil above Fin Wall (Soil-soil)

Value given by geotech
Friction Force due to soil block above 1 Fin Wall= 27 kips — =divide by pile nos. 4 kips per pile — Factored (x 1.4) 5

3. Max. DD at pile tip elevation

Value estimated from A-pile charts

from DD load at pile tip elevations
Refer to geotech Apile charts

Pile tip elevation= 66 ft — For Abutment 1
Max DD at that elev. 250 kips — For Abutment 1 — x1.4= 350
Pile tip elevation 65 ft — For Abutment 2
Max DD at that elev. 275 kips — For Abutment 2 — x1.4= 385

COWI	PROJECT:	I-405 R2B Final Design - BR 28W (112th Ave)	JOB NO.:	A207833
	SUBJECT:	Pile Loadings and Analysis - Load Estimates	Sheet:	1
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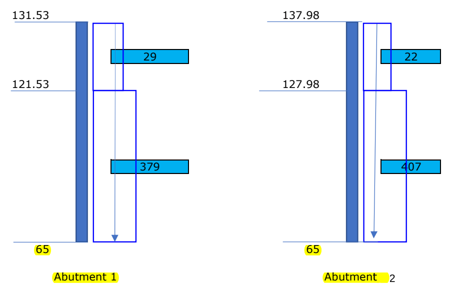
DOWNDRAG PROFILE*

*for EXT Case, use 1.0 factor instead of 1.4



from Fin wall and Pile Cap

from Fin wall + Pile Cap + soil in contact with pile

Soil Deformation Profile - For both abutments

No deformation if external casing is used

agreed with geotech and wall designer

Load Cases - Axial Demands*

*lateral loads refer to next sheet

	For Pile nos. and lengths		Abutment 1	Abutment 2	
1	Factored DL + Factored LL	STR I	441	509	
2*	Factored DL + Factored LL + additional axial from EQ	EXT I	351	370	*additional axial load added due to lateral loads to EXT case
	For Structural Checks				
	- Top 20ft of Piles - governed by lateral loads		Abutment 1 Axial	Abutment 2 Axial	
1	Factored DL + Factored LL + lateral loads	STR I	441	509	
2*	Factored DL + Factored LL + lateral loads	EXT I	351	370	*additional axial load added due to lateral loads to EXT case
3	Factored DL + Factored DD + lateral loads	STR	316	358	
4	Factored DL + Factored DD + lateral loads	EXT	327	341	
	- After 20ft of Piles - governed by axial loads*				
1	Factored DL + Factored LL + lateral loads	STR I	441	509	
2*	Factored DL + Factored LL + lateral loads	EXT I	351	370	*additional axial load added due to lateral loads added
3	Factored DL + Factored DD + lateral loads	STR	666	743	
4	Factored DL + Factored DD + lateral loads	EXT	577	616	

=values used for design check

COWI		
PROJECT:	I-405 R2B Final Design - BR 28W (112th Ave)	JOB NO.: A207833
SUBJECT:	Pile Structural Checks	Sheet: 2
BY:	RSGR	DATE: 9/13/2021
CHECK:	MEDN	DATE: 9/13/2021

Procedure:
- determine number of piles required to resist vertical demands, based on geotech vertical load capacity
- estimate lateral loads and moments
- Check effects of loads on structural capacity of selected pile using Lpile

1.0 Vertical Demands

	Unfactored DL [kip]	Str Case I Vertical [kip]	Ext Case I Vertical [kip]
Abut West	1496	3084	1895
Abut East	1516	3054	1858

Estimated in Load estimation sheet from previous sheet

Abutment Length

Abut West	51.21	ft
Abut East	54.75	ft

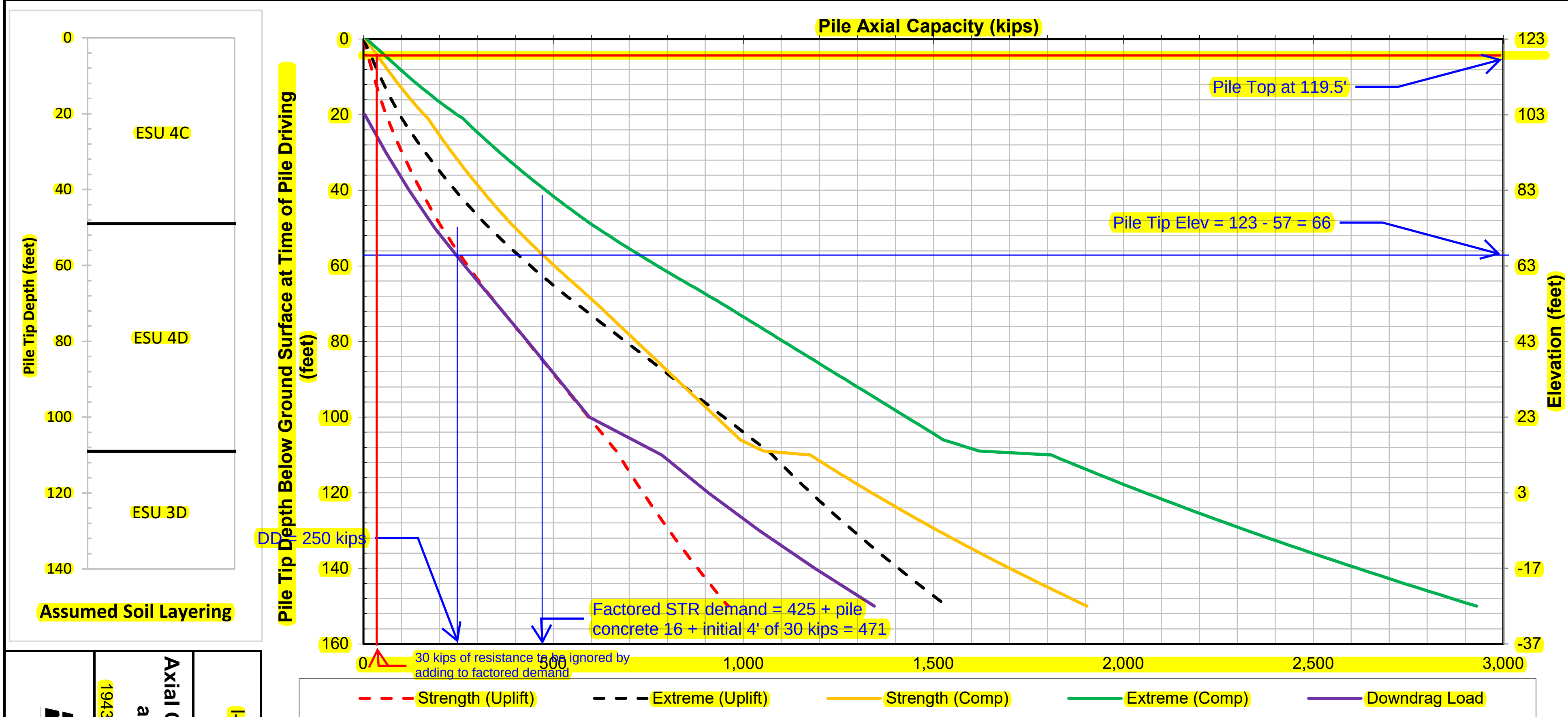
Taken from the latest drawing file

Pile Diameter; D	2	ft	
Average Spacing Abut 1	6.54	ft	WSDOT GDM 8.12.2.2; 2.5D satisfied
Average Spacing Abut 2	9.25	ft	
West Pile embed Length	64	ft	Min. required length to meet axial demand
East Pile embed Length	64	ft	

2.0 Estimate Piles Required

	Piles Req'd [ea]	Piles prov'd [ea]	Length (ft)	C/C outside piles [ft]	Str I Load per Pile	Ext I Load per Pile	Fit in pile cap	
								Nos. and Length Estimated based on geo tech info in A Pile Diagrams
Abut West	6	7	64	39	441	351	OK	OK. number of piles required will fit with abutment length
Abut East	6	6	64	46	509	370	OK	

Compare against axial capacity chart on next page



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I-405 Renton to Bellevue Express Toll Lanes
Bellevue, WA

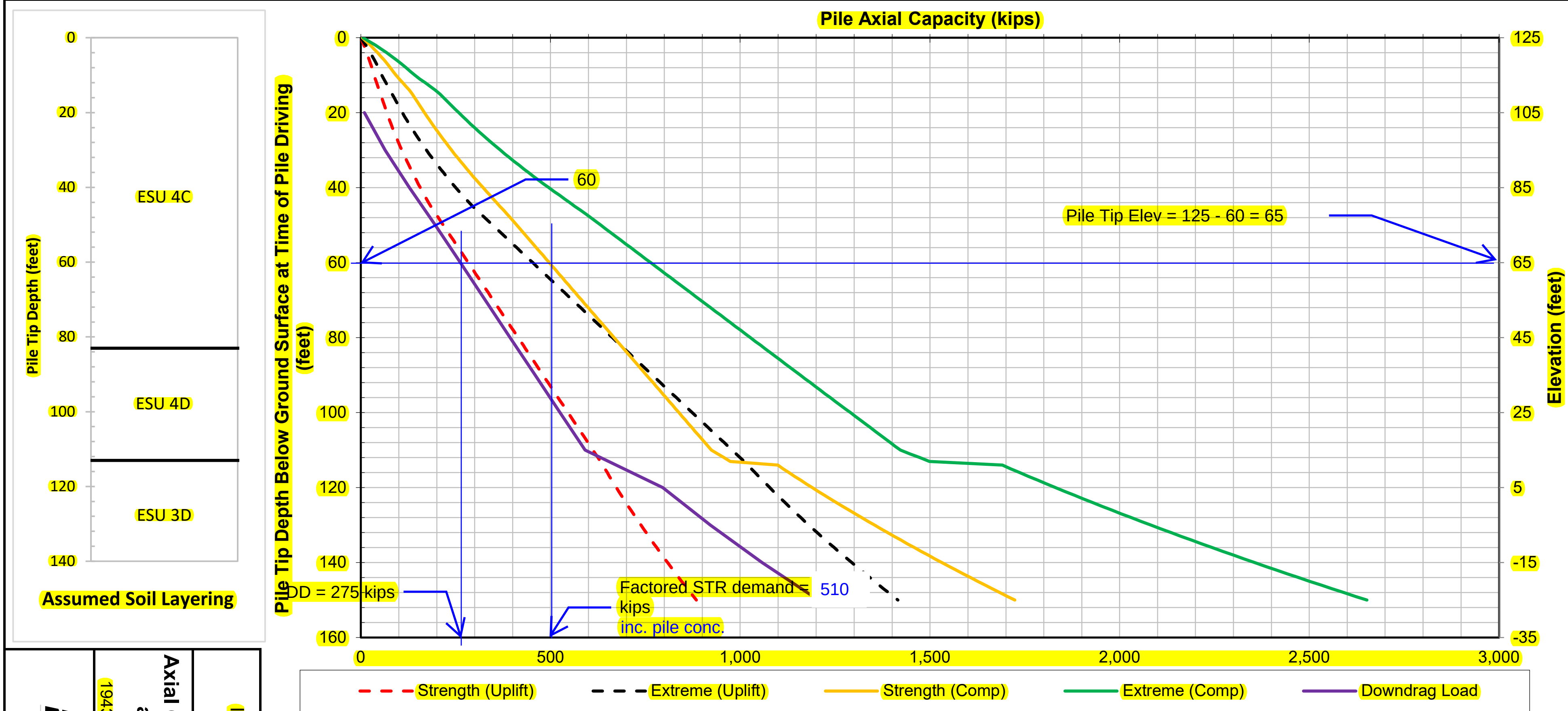
Axial Capacity for Driven Open-Ended Pipe Piles at Bridge 28W West Pier - 24-inch O.D.

19434-04
12/2020

Figure
H-1

Notes:

1) A steel elastic modulus of 29,000,000 psi and pile thickness of 0.625-inch was assumed in computations.
2) A ground surface elevation at the time of pile driving of approximately 123 feet (NAVD88) was assumed.
3) The capacities presented do not incorporate a downdrag load. Downdrag load and depth of downdrag considered is dependent on pile embedment length. The downdrag depth and magnitude should be chosen from the "Downdrag Load" series that corresponds to the total pile embedment.
4) The downdrag load is dependent on dead load applied at the pile head, and was calculated using a conservatively low value of 240 kips. The downdrag load was calculated using the neutral plane methodology as described in FHWA-NHI-16-009 using APILE.
5) No load factors were applied to the "Downdrag Load" series shown. Apply appropriate load factor to the downdrag load according to Table 3.4.1-2 in AASHTO LRFD 8th Edition (2017). The "Tomlinson Method" should be used in reference for native soils and "O'Neill and Reese (2010) Method" for the MSE wall.
6) Recommended resistance factors for compression and uplift resistance are 0.65 and 0.5, respectively, and were applied for the Strength Limit State according to AASHTO LRFD 8th Edition (2017). This assumes pile capacity verification by dynamic testing with signal matching.
7) For the Extreme Event Limit State, recommended resistance factors are 1.0 and 0.8 for compression and uplift, respectively.
8) For pile penetrations less than 20 feet, the nominal resistance is less than the estimated lower bound dead load at the top of pile. Embed piles deeper than 20 feet.
9) To account for side resistance along the length of the pile through the MSE wall, see chart in Figure H-5. Magnitude of skin resistance is dependent on wall height.
10) If piles are installed without casing before MSE wall construction, downdrag will need to be considered within the MSE wall in addition to what is shown on the "Downdrag Load" series. See chart in Figure H-5 for skin resistance in the MSE.
11) Per GDM Section 8.12.2.1, the maximum nominal pile resistance for 24-inch piles is 1,500 kips. The nominal pile capacity may be increased if a pile load test is performed.



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6) Recommended resistance factors for compression and uplift resistance are 0.65 and 0.5, respectively, and were applied for the Strength Limit State according to AASHTO LRFD 8th Edition (2017). This assumes pile capacity verification by dynamic testing with signal matching.
7) For the Extreme Event Limit State, recommended resistance factors are 1.0 and 0.8 for compression and uplift, respectively.
8) For pile penetrations less than 20 feet, the nominal resistance is less than the estimated lower bound dead load at the top of pile. Embed piles deeper than 20 feet.
9) To account for side resistance along the length of the pile through the slope fill, see chart in Figure H-6. Magnitude of skin resistance is dependent on wall height.
10) If piles are installed without casing before fill slope construction, downdrag will need to be considered within the slope fill in addition to what is shown on the "Downdrag Load" series. See chart in Figure H-6 for skin resistance in the fill slope.
11) Per GDM Section 8.12.2.1, the maximum nominal pile resistance for 24-inch piles is 1,500 kips. The nominal pile capacity may be increased if a pile load test is performed.

I-405 Renton to Bellevue Express Toll Lanes
Bellevue, WA

Axial Capacity for Driven Open-Ended Pipe Piles at Bridge 28W East Pier - 24-inch O.D.

19434-04

12/2020

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Figure
H-2

3.0 Total Seismic lateral loads on Piles: Transverse only (EQ)

	Abut West	Abut East		
F_{PGA}	1.17	1.17		AASHTO Table 3.10.3.2-1
PGA	0.43	0.43		AASHTO Fig 3.10.2.2-1
Seismic acceleration, As=	0.503	0.503	$=F_{PGA}PGA$	AASHTO 3.10.4.2-2
Tributary weight, M=	1496	1516	Kips	inc. weight of substructure (234 kips for abut 1 and 270 kips for abut 2)
Seismic demand, S=	706	708	Kips	(for substructure weight 0.6xAs is used)

At abut 1, calculate resistance from 2 Fin Wall

Resistance offered by Fin Wall is estimated by the geotechnical engineer.

Refer to geotech addendum for recommendations

Piles are designed for the case when Fin wall offers the least resistance. i.e. 200 kips

This accounts for active+passie pressures on fin walls and MSE wall

Minimum 0.5" of Fin wall movememnt is req. for 100% mobilization of Fin wall resistance of 200 kips

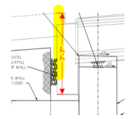
Resistance from walls per geotech calculations, R=	200	kips
Limit R for pile design=	200	kips

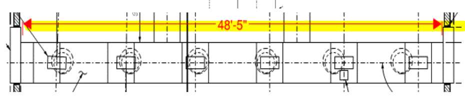
At abut 1, calculate slide resistance from friction between concrete and soil

C=	0.8		
Resistance Factor, φ=	0.9		
Φ'F=	25.46		=2/3 of angle of friction of 38 degrees
TAN δ=TAN $Φ'_F$=	0.48		

V=Lateral earth pressure

Seismic Active earth pressure coefficient, Kae=	0.42		
Height of diaphragm below ground engaged, Hw=	7.6	ft	
Length of diaphragm, Lw=	48.42	ft	
soil unit weight, gamma=	0.135	kcf	
Pressure at the bottom, pp=	0.43	ksf	=Kae x gamma x Hw
Total force, V=	37	kips	=0.5 x pp x 3.5 of pile cap x Lw
factored Friction Resistance, Rr=	13	kips	=phi x CV x TAN Φ'F





If the soil beneath the footing is cohesionless, the nominal sliding resistance between soil and foundation shall be taken as:

$$R_\tau = CV \tan \phi_f \quad (10.6.3.4\text{-}2)$$

C = 1.0 for concrete cast against soil
= 0.8 for precast concrete footing

ϕ_f = internal friction angle of drained soil (degrees)
V = total vertical force (kips)

Hence, seismic total seismic demand in transverse direction for abutment 1:

Net Seismic Force = S - R - Rr =	493	kips
per pile =	70	kips

At abut 1, increase in longitudnal force due to passive force in Fin Walls

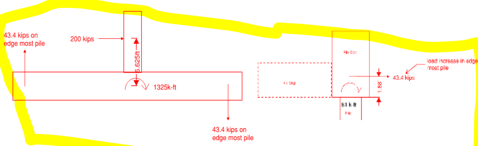
*The increase in longitudnal lateral force due to passive force in Fin Wall; is marked as X in the figure
Here, assumed 200 kips of passive force is used for the Fin Wall

Moment by passive force, Mp= 1325 k-ft =(200)*(Fin wall length*0.5+0.5*pile cap width)
=calculate increase in longitudnal force due to this moment (next page); Note that this is on the conservative side since

Increase in lateral force, X= (refer to figure on the side)

Pile Spacing, s= 6.54 ft

x	d (from centre)		Force, (Mnet x d / (Σd^2)), kips
1	6.54	ft	14.5
3	13.08	ft	28.9
5	19.62	ft	43.4
Σd^2=	599		



Hence, max longitudinal force going to pile due to transverse EQ=	43.4	kips	Add this to longitudnal lateral force for EQ case for abut 1 piles only
Associate longitudinal moment at top of edge pile=	81.6	k-ft	

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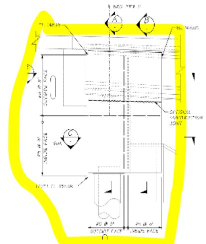
At abut 2, calculate passive resistance from curtain Wall

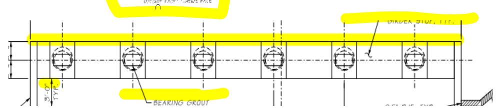
Length of curtain wall engaged, Lw=	7.0	ft	
Height of curtain wall engaged, Hw=	10.24	ft	
Depth below ground, z=	10.24	ft	
Passive Pressure developed during seismic event pp=	6.83	ksf	=0.67 x Hw
Passive Force developed, Pp=	343	kips	=pp x z x Lw x 0.7



At abut 2, calculate slide resistance from friction between concrete and soil

C=	0.8
Resistance Factor, φ =	0.9
Φ'F=	25.46
TAN δ=TAN Φ'F=	0.48

V=Lateral earth pressure



Seismic Active earth pressure coefficient, Kae=	0.42		
Height of pile cap and diaphragm engaged, Hw=	11.5	ft	
Length of pile cap engaged, Lw=	54.75	ft	
soil unit weight, gamma=	0.135	kcf	
Pressure at the bottom, pp=	0.65	ksf	=Kae x gamma x Hw
Total force, V=	205	kips	=0.5 x pp x Hw x Lw
factored Friction Resistance, Rr=	70	kips	=phi x CV x TAN Φ'F

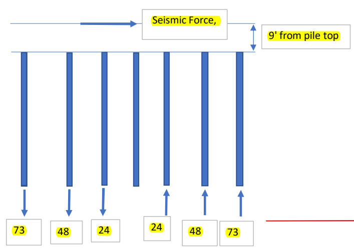
Hence, seismic total seismic demand in transverse direction for abutment 2 assuming that only 85% of passive pressure is developed.

Net Seismic Force = S - 0.85Pp - Rr =	347	kips
per pile =	58	kips

	Abut West	Abut East	
Reduced seismic demand, S=	493	347	kips
per pile=	70	58	kips

These values are for Transverse Direction, Use East abutment values for analysis/design

4.0 Pile Axial Loads increment due to lateral Seismic Force



Calculate Moment Arm at abutment 2

	Lateral Force, L	act at, D	LD
Superstructure	686	8	5486
Pile Cap	125	3.15	393
Fin Wall	8	1.84	15
Passive Resistance from Fin Wall	-200	1.84	-367

				D eff	
Σ	618		5527	9	=ΣLD/ΣL

Added back to EXT case

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5.0 **Seismic lateral loads on superstructure: Longitudnal only (EQ)**

	Abut West	Abut East	
F_{PGA}	1.17	1.17	
PGA	0.43	0.43	
Seismic acceleration, As=	0.503	0.503	$=F_{PGA}$PGA
Tributary weight, M=	1264	1250	Kips excluding pile cap weight and shear keys and Fin Wall
Lateral force=	636	629	kips
Lateral force, S=	1265	kips	

Hence total lateral force, S = 1265 kips

This is to be resisted by passive pressure behind abut diaphragm as shown below:

Check if West diaphragm can develop passive resistance to take seismic force

Length of diaphragm between curtain walls, Ld =	48.8	ft	
Height of ground at end of diaphragm, Hd =	7.60	ft	Required Height
Passive pressure Force developed, Pp=	1322	kips	=0.67 x Hd x Hd x Ld x 0.7
Passive force > Total lateral seismic force (1246 kips)	**ok**		

- For cohesionless, nonplastic backfill (fines content less than 30 percent), the passive pressure pp may be assumed equal to 2Hw/3 ksf per foot of wall length.

Hence, ~ 2.8' (2'-10") below the bottom flange of the girder (instead of 1')

Superstructure Depth	4.9	ft	
Required Height	7.60	ft	
Longitudnal Slope effect	0.1	ft	
Diaphragm extension below bottom flange	2.8	ft	=(7.6 - 4.9 + 0.1)

Assuming dense sand,
the following displacement is needed to develop
passive pressure

Δ/H=	0.01	
Hd=H=	7.6	ft
Δ=	**0.91**	inches

this movement is less than the 1.5" gap between end diaphragm and pile c

Table C3.11.1-1—Approximate Values of Relative Movements Required to Reach Active or Passive Earth Pressure Conditions (Clough and Duncan, 1991)

Type of Backfill	Values of Δ/H Active	Passive
Dense sand	0.001	0.01
Medium dense sand	0.002	0.02
Loose sand	0.004	0.04
Compacted silt	0.002	0.02
Compacted lean clay	0.010	0.05
Compacted fat clay	0.010	0.05

Hence, for piles, seismic force in long. direction comes from Seismic active pressure behind the pile cap and interia of pile cap self weig

Check if East diaphragm can develop passive resistance to take seismic force

Length of diaphragm, Ld =	54.75	ft	
Height of ground at end of diaphragm, Hd =	7.60	ft	Required Height
Passive pressure Force developed, Pp=	1483	kips	=0.67 x Hd x Hd x Ld x 0.7
Passive force > Total lateral seismic force (1246 kips)	**ok**		

Hence, use 2.8' (2'-10") below the bottom flange of the girder (instead of 1')

Superstructure Depth	4.9	ft	
Required Height	7.60	ft	
Longitudnal Slope effect	0	ft	
Diaphragm extension below bottom flange	2.7	ft	=(7.6 - 4.9 + 0)

Assuming dense sand,
the following displacement is needed to develop
passive pressure

Δ/H=	0.01	
Hd=H=	7.6	ft
Δ=	**0.91**	inches

this movement is less than the 1.5" gap between end diaphragm and pile cap

Hence, for piles, seismic force in long. direction comes only from seismic active pressure behind the pile cap and interia of pile cap self weight

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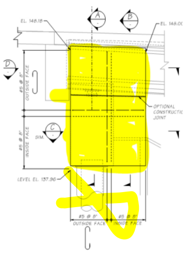
6.0 Lateral Demands on Piles : Bearing shear deformation (Londitudnal) (FR)

	Abut West	Abut East		
Max. bearing deformation=	1.5	1.5	in	
G=	0.165	0.165	ksi	Shear modulus
A=	210	210	in2	area of elastomeric pad
height of pad=	2.5	2.5	in	
Max. Force P=	20.79	20.79	kips	=This is the max bearing force
No. of Girders=	6.0	6.0		
Force per pile=	17.82	20.79	kips/pile	
Induced Moment at pile=	107	125	k-ft/pile	6ft moment arm

Do not consider FR when deflection of pile top with FR is > 1.5" at which FR will help



7.0 Lateral Demands on Piles : Wind (transverse) (WS)

Curtain Wall height	10.000	ft
Length	7.000	ft

Wind pressure on abutment

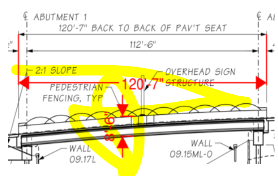
Limit State	PZ
Strength V	0.050

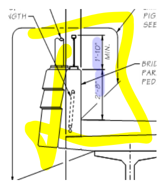
Wind load on abutment

Limit State	Force	
Strength V	3.5	kips/abutment

Wind pressure on superstructure

Wind pressure (ksf)	
	WEC "C"
Limit State	Transv
Strength V	0.040





Wind load on Superstructure

height	10.333	ft
Length	120.583	ft

Limit State	Force	
Strength V	49.8	kips
	24.9	kips/abutment

Total Wind Load

Strength V	28.4	kips/abutment
	5	kips/pile

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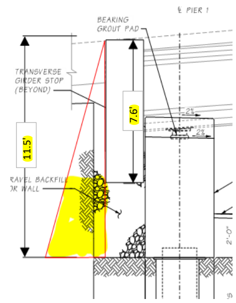
8.0 Lateral Demands on Piles : earth pressure on pile cap (EH) - long direction - STR

Soil unit weight	0.135	kcf
Gravel Backfill Ko	0.412	at rest - STR case
Gravel Backfill Ka	0.235	Active - STR case
Length of pile cap	54.75	ft (use biggest abutment)



LEP values

below ground	at rest	active
ft	k/sqf	k/sqf
0	0.00	0.00
7.6	0.42	0.24
11.5	0.64	0.36

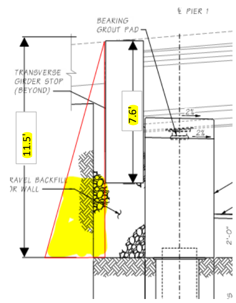
Total force on pile cap (kips)

Area of highlighted portion x length of pile cap

	Use Active Pressure Force	
Total=	65	kips
per pile=	11	kips
Moment arm=	2.5	ft
Induced moment=	27	kips-ft/pile

9.0 Lateral Demands on Piles : Seismic earth pressure (EH) - long direction - EQ

Seismic Active earth pressure coefficient, Kae= 0.420



LEP EQ values

below ground	earthquake
ft	k/sqf
0	0.00
7.6	0.43
11.5	0.65

Total force on pile cap (kips)

Area of highlighted portion x length of pile cap

	Seismic Active	
LEP:EQ force	115.6	kips
per pile, Abut 2	19.3	kips

Seismic Horizontal Acceleration coefficient, 0.6 x k_h=	0.302	g
Weight of pile cap, Ww=	278.00	kips
Horizontal inertial force due to seismic loading, P_{IR} =	83.9	kips
Horizontal inertial force due to seismic loading, P_{IR} =	14.0	kips/per pile

Effective Lateral Seismic Force, P_{EQ}=	26	kips/per pile
Moment arm=	2.5	ft
Induced moment=	66	kips-ft/pile

=MAX(0.5*Pir+LEP,0.5*LEP+Pir)

8.0 Load combinations - Abutment 1

For Lateral Loads	EH	FR	EQ	WS
Strength I Load Case:	1.5	1	0	0
Strength V Load Case:	1.5	1	0	1
Extreme I Load Case:	1	1	1	0
Service I Load Case:	1	1	0	1

With FR

		Axial Load at top	Long	Trans	Check Design Moment at Pile Top	
	Strength I Load Case:	441	34	0	165	
Case 1	Strength V Load Case:	441	34	5	165	
Case 2	Extreme I Load Case:	351	72	70	208	(30% L + 100%T)
Case 3	Extreme I Load Case:	351	60	21	197	(100% L + 30%T)
	Service I Load Case:	315	32	5	152	

Without FR

		Axial Load at top	Long	Trans	Check Design Moment at Pile Top	
	Strength I Load Case:	441	25	0	103	0.5 FR Used in STR Case
Case 4	Strength V Load Case:	441	25	5	103	0.5 FR Used in STR Case
Case 5	Extreme I Load Case:	351	51	70	101	(30% L + 100%T)
Case 6	Extreme I Load Case:	351	39	21	90	(100% L + 30%T)

Do not consider FR when deflection of pile top with FR is > 1.5" at which FR will help

9.0 Load combinations - Abutment 2

	EH	FR	EQ	WS
Strength I Load Case:	1.5	1	0	0
Strength V Load Case:	1.5	1	0	1
Extreme I Load Case:	1	1	1	0
Service I Load Case:	1	1	0	1

With FR

		Axial Load	Long	Trans	Check Design Moment at Pile Top	
	Strength I Load Case:	509	34	0	165	
Case 1	Strength V Load Case:	509	34	5	165	
Case 2	Extreme I Load Case:	370	29	58	144	(30% L + 100%T)
Case 3	Extreme I Load Case:	370	47	17	190	(100% L + 30%T)
	Service I Load Case:	364	32	5	152	

		Axial Load	Long	Trans	Check Design Moment at Pile Top	
	Strength I Load Case:	509	16	0	40	No FR Used in STR Case
Case 4	Strength V Load Case:	509	16	5	40	No FR Used in STR Case
Case 5	Extreme I Load Case:	370	18	58	82	(30% L + 100%T) 0.5 FR Used in ETR Case
Case 6	Extreme I Load Case:	370	26	17	66	(100% L + 30%T) No FR Used in ETR Case

Do not consider FR when deflection of pile top with FR is > 1.5" at which FR will help

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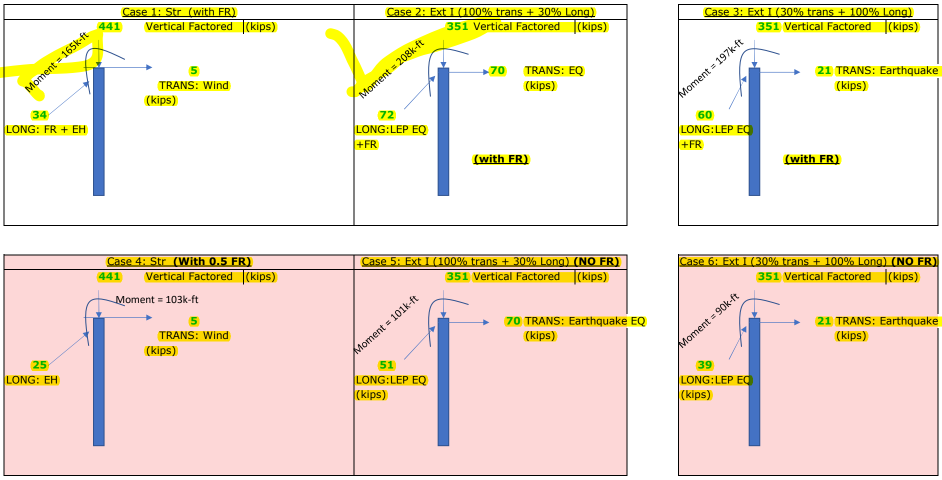
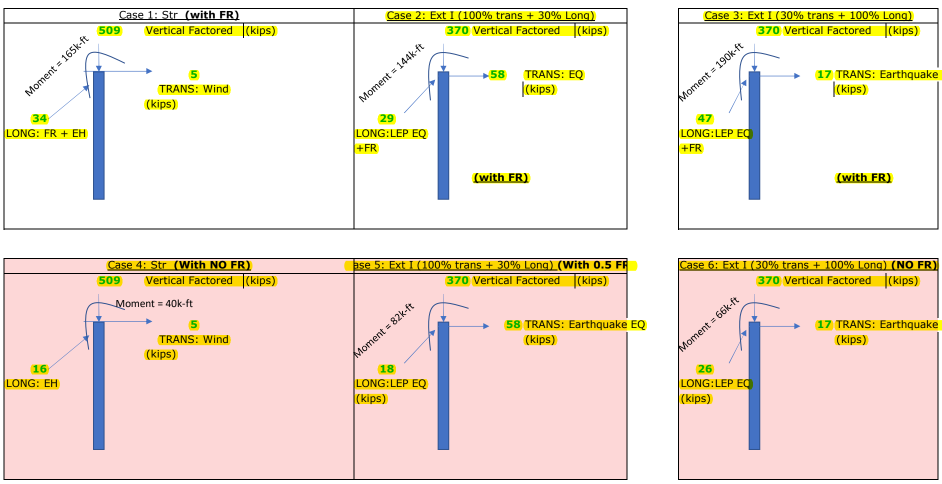
10.0 Load Cases to analyze in L Pile (kips) - Abutment 1



Cases checked for: Case 1 2 3 are ignored as including bearing deformation causes substructure/pile to move 1.5+ inches at which FR will help. (Long Direction)
Hence, Case 4 5 6 are selected instead where deflection is between 1.2" for STR and 2.5" for EXT cases (Long Direction)

11.0 Load Cases to analyze in L Pile (kips) - Abutment 2



Cases checked for: Case 1 2 3 are excluded as bearing deformation causes substructure/pile to move 1.5+ inches at which FR will exert load. (Long Direction)
Hence, Case 4 5 6 are selected instead where deflection is between 1" for STR and 1.6" for EXT cases (Long Direction)

P-y modification factor required for long analysis:

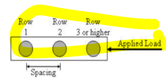
	Row 1	
Pile CTC spacing 4B	0.9	both abutment

Pile Top = Pinned

P-Y modification factors in trans using linear interpolation from following figure:

Table 10.7.2.4-1—Pile P-Multipliers, P_m, for Multiple Row Shading (averaged from Hannigan et al., 2006)

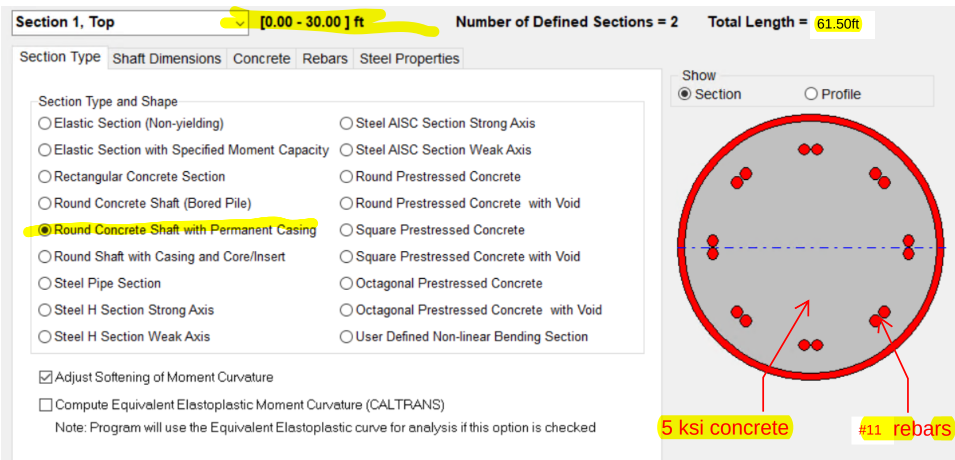
Pile CTC spacing (in the direction of loading)	P-Multipliers, P_m Row 1	Row 2	Row 3 and higher
3B	0.8	0.4	0.3
5B	1.0	0.85	0.7

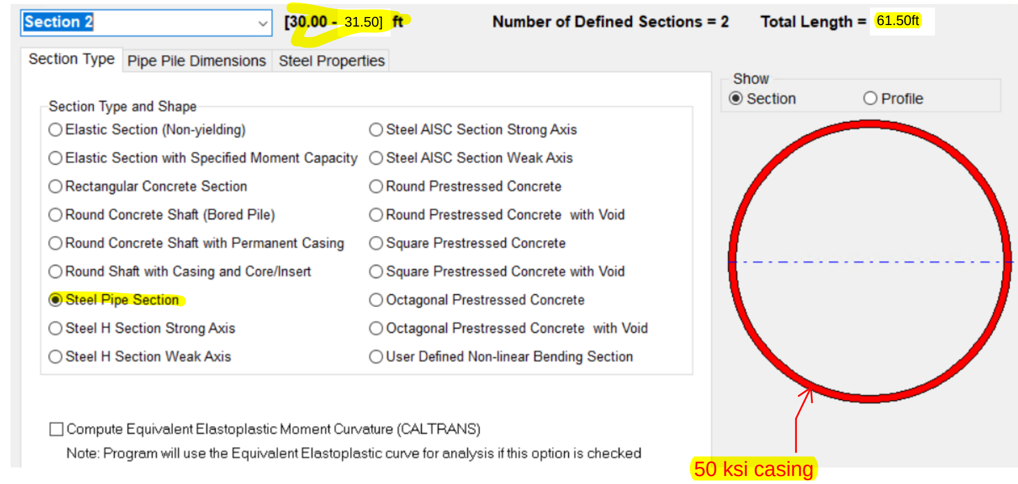


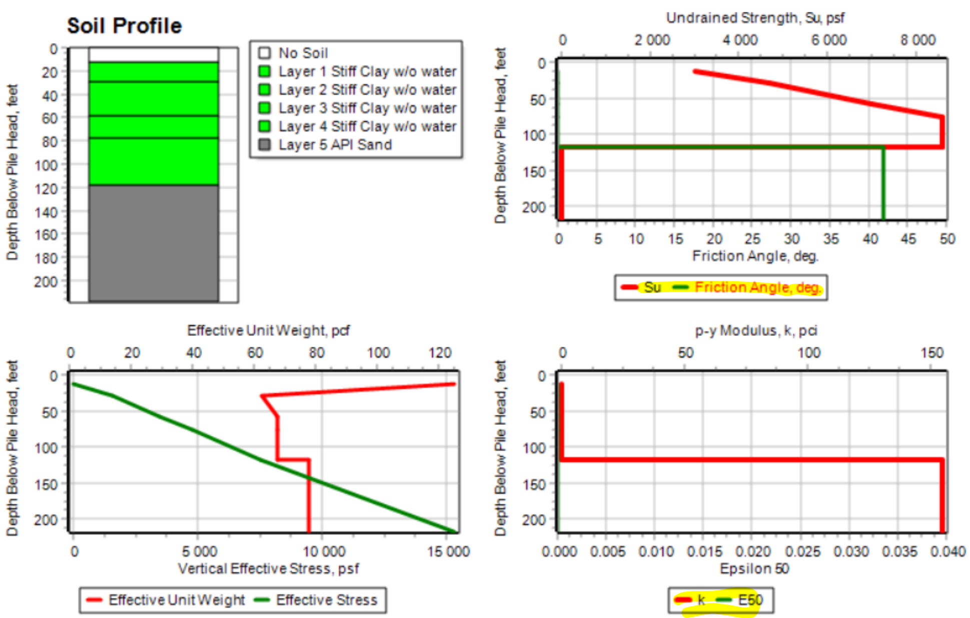
	Row 1	Row 3 +	
Pile CTC spacing 3.3B	0.83	0.35	abut 1
Pile CTC spacing 4.625B	0.96	0.6	abut 2

Pile Top = Fixed

L PILE INPUT FOR ABUTMENT 1 (WEST)

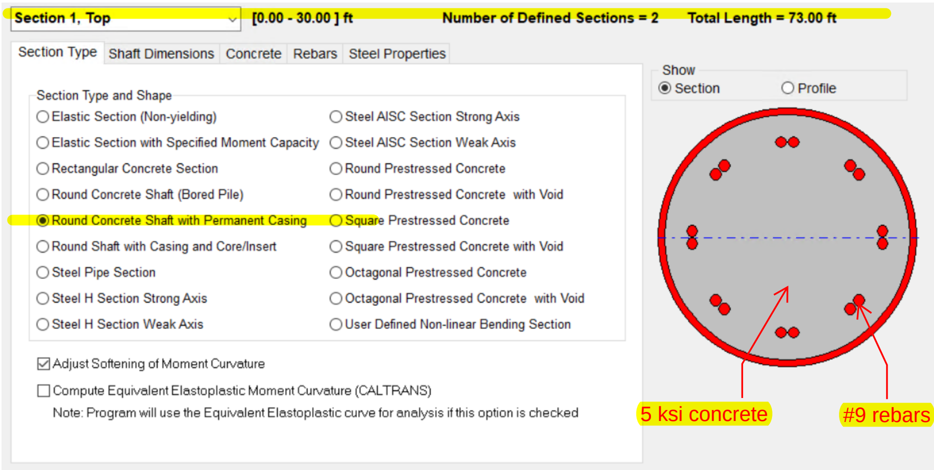


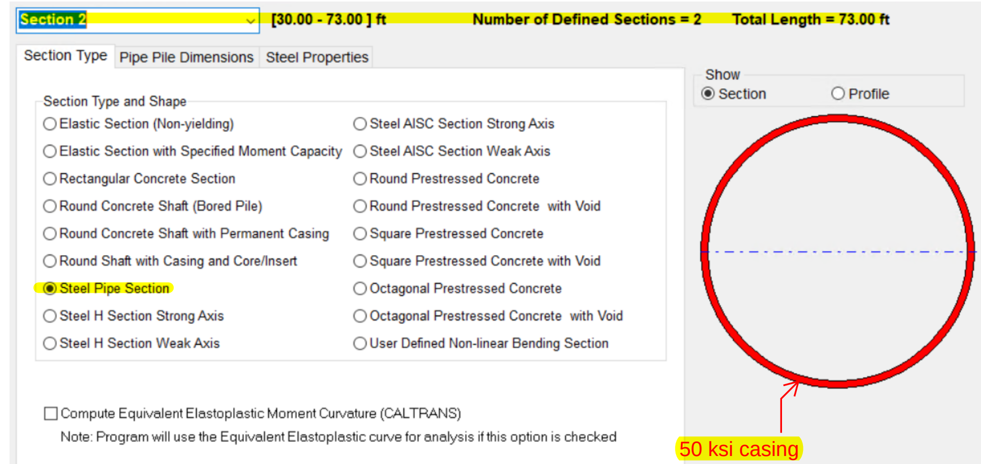


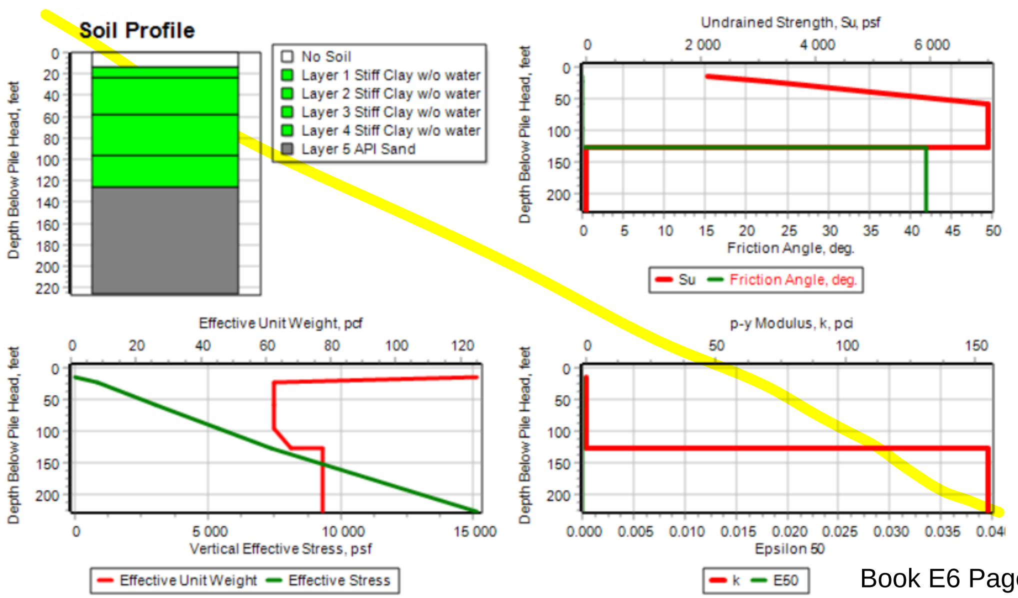


Soil layer input

L PILE INPUT FOR ABUTMENT 2 (EAST)



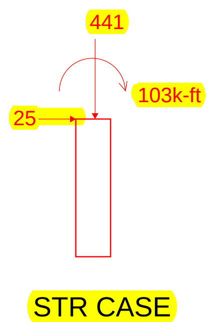


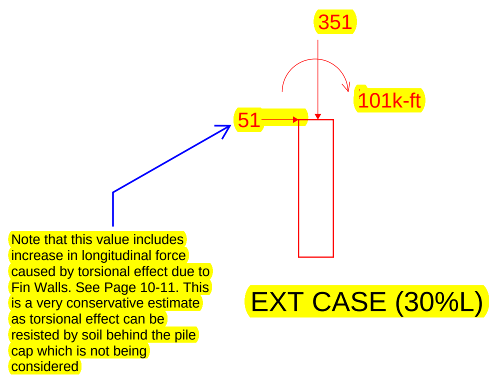


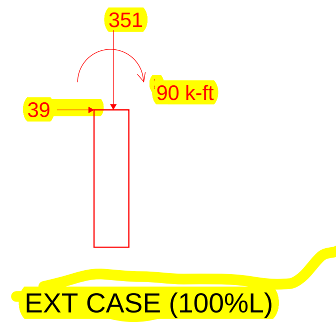
ABUTMENT 1 WEST

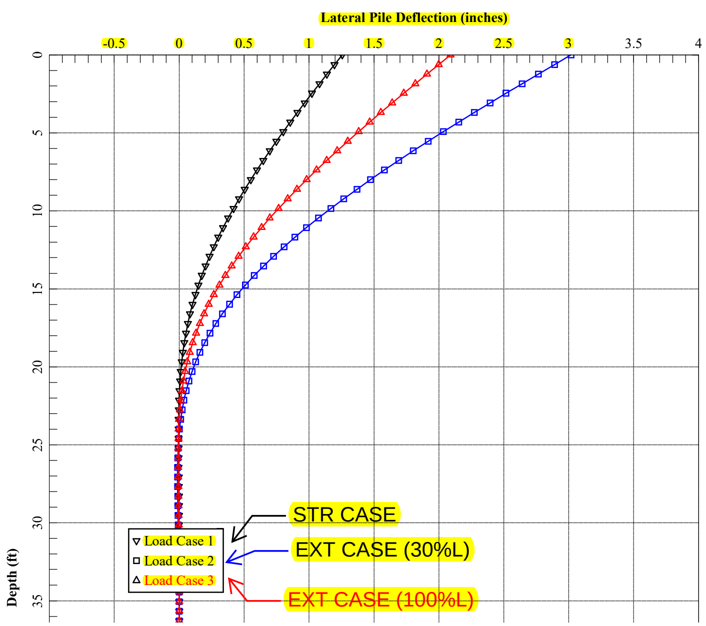
LONGITUDNAL CASES ANALYSED (STR Case and EXT case)

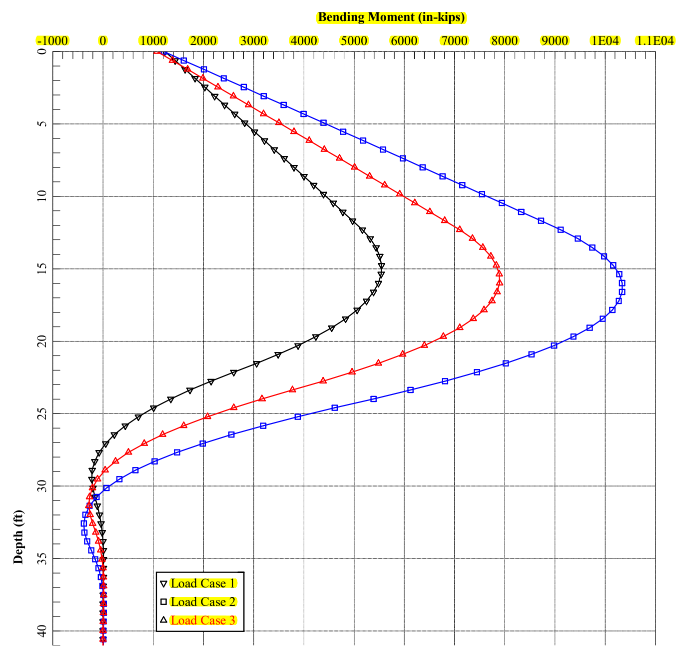
LONG CASE
- Pin Head
- 0.9 p-y multiplier

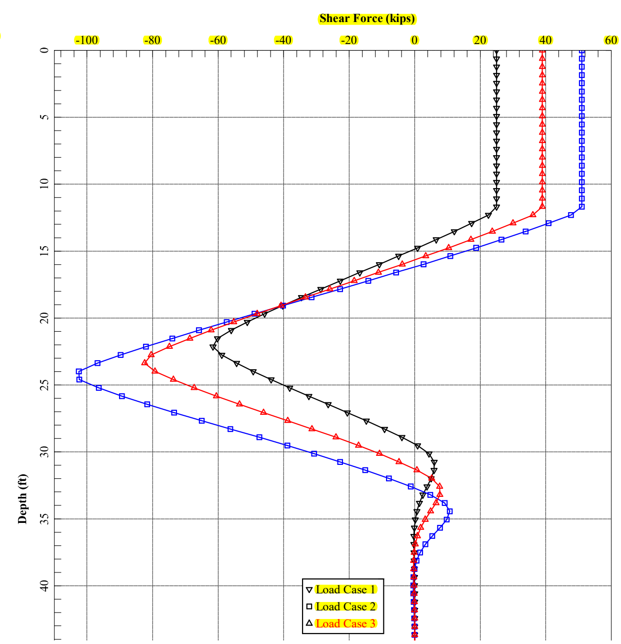








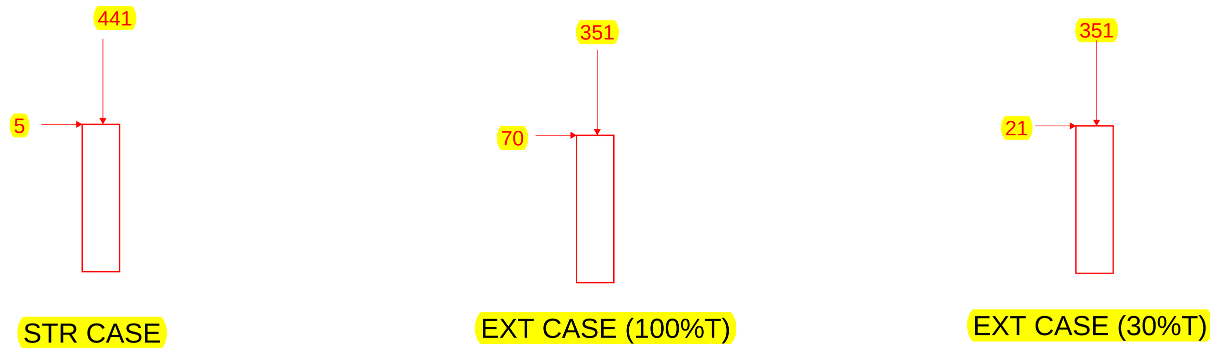


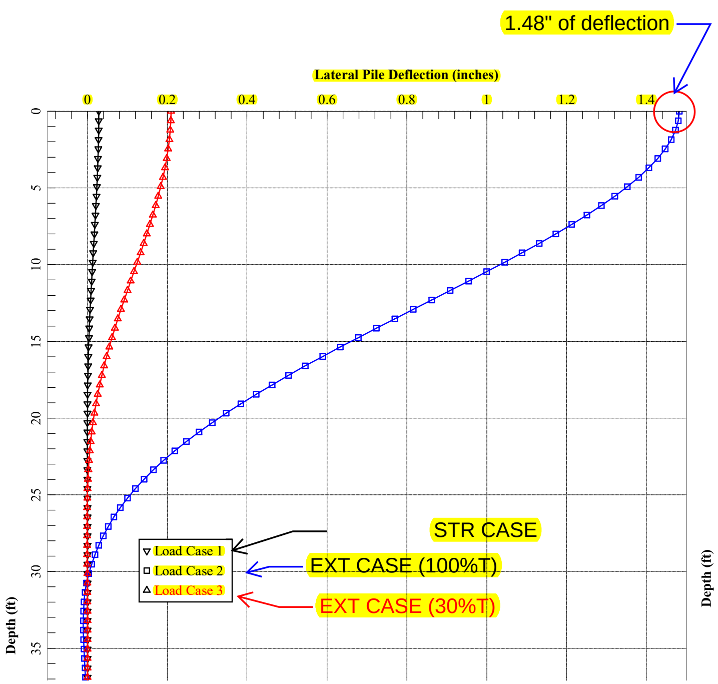


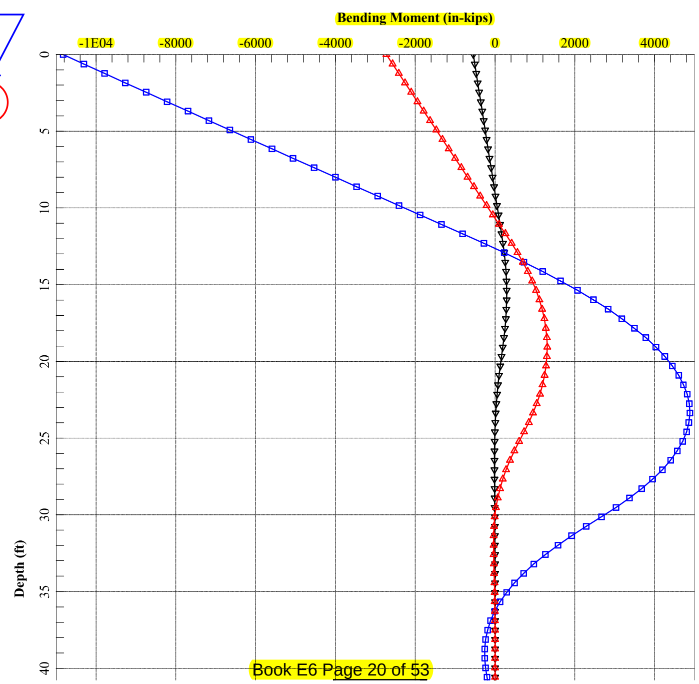
ABUTMENT 1 WEST

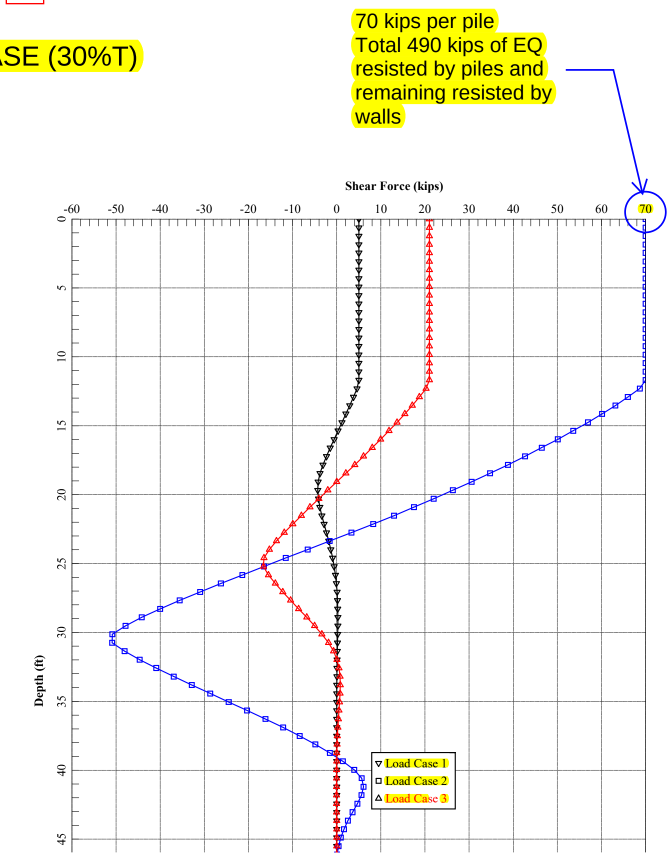
TRANSVERSE CASES ANALYSED (STR Case and EXT case)

TRANS CASE
- 0.35 p-y multiplier
- fixed pile head





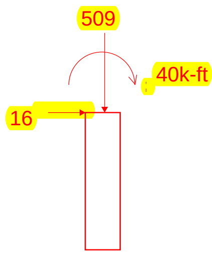




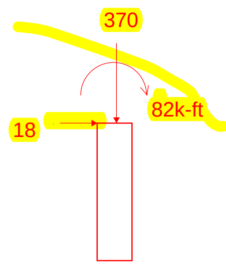
ABUTMENT 2 EAST

LONGITUDNAL CASES ANALYSED (STR Case and EXT case)

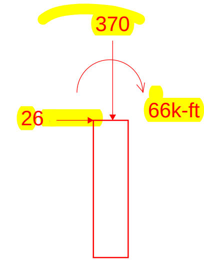
LONG CASE
- Pin Head
- 0.9 p-y multiplier



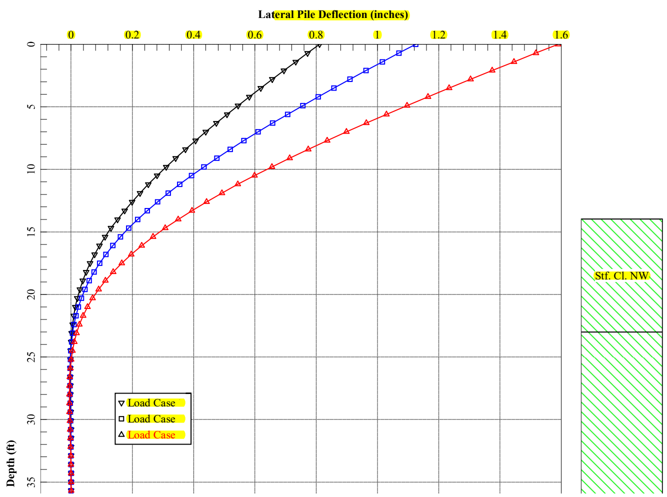
STR CASE

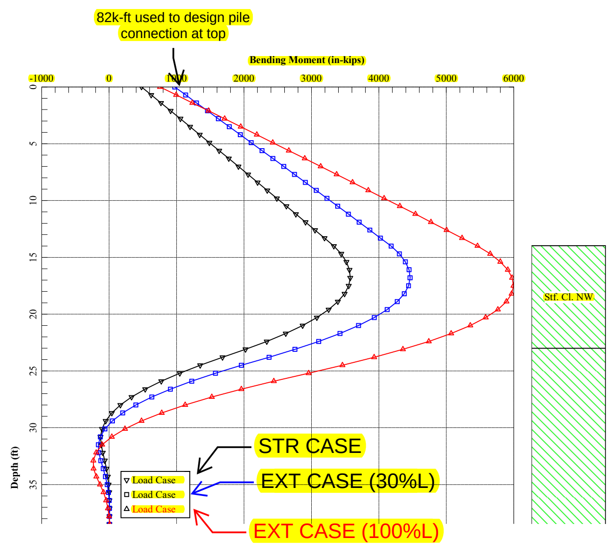


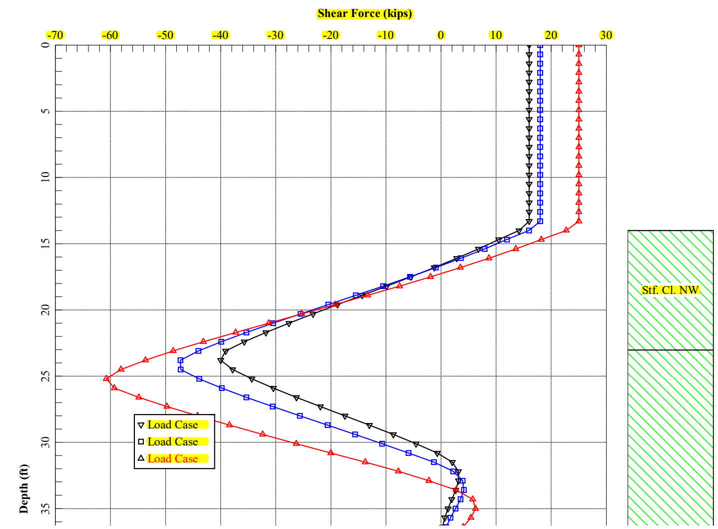
EXT CASE (30%L)



EXT CASE (100%L)



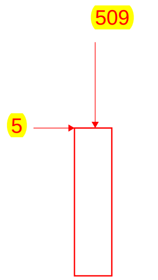




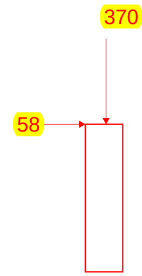
ABUTMENT 2 EAST

TRANSVERSE CASES ANALYSED (STR Case and EXT case)

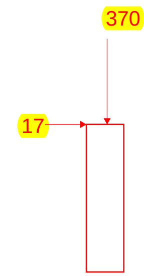
TRANS CASE
- 0.56 p-y multiplier
- Fixed pile head



STR CASE

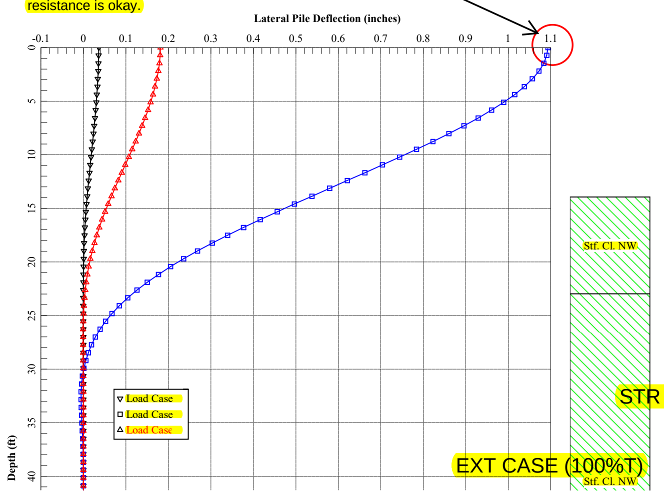


EXT CASE (100%T)

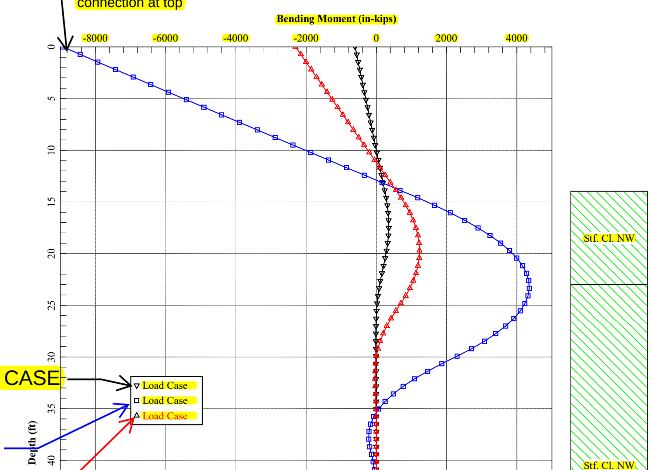


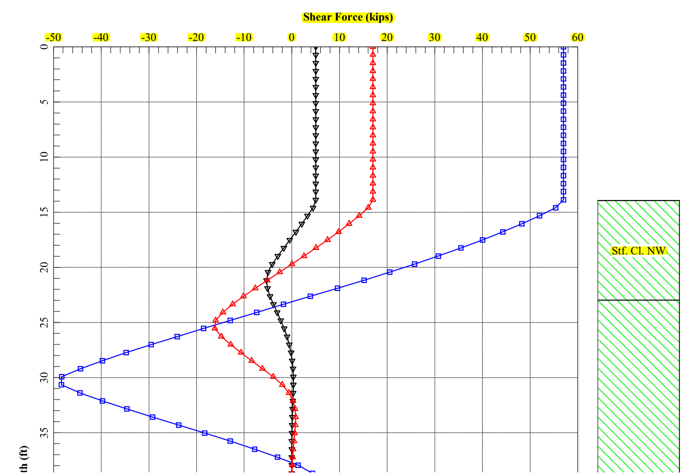
EXT CASE (30%T)

Top deflection = 1.1" which is around 90% of deflection needed for full mobilization of passive pressure. Assuming dense sand, deflection needed for full mobilization of passive pressure is = 0.01 x 10.24' x 12" = 1.23", where 0.01 is from AASHTO table C3.11.1 and 10.24' is the height of curtain wall and soil behind it, Hence our assumption of 85% effectiveness of passive resistance is okay.



700k-ft used to design pile connection at top





STR CASE

EXT CASE (100%T)

EXT CASE (30%T)

11.0 Lateral Pile Check using L- Pile 0.625" THK.

					Outside Diameter after corossion = 23.850 in	
Diam	24.0	in			Inside Diameter after corossion = 22.750 in	
Thickness	0.625	in				
Corossion outside	0.075	in			Corrosion Rate 0.001 from Table 1: WSDOT Memo WSDOT BDM 6.7; Used 0.0015 rate for first 15ft	
Corossion inside	0.000	in			No Inside corossion required	
Net Thickness	0.5500	in				
Area; Ag	40.3	in2			gross area after corossion	
S	229.2	in3			Elastic Modulus; corroded section	
I	2733.6	in4			Moment of Interia; corroded section	
E	29000.0	ksi			Youngs Modulus	
Fy	50.0	ksi			ASTM A252 Grade 3 Pipe	
Fye	55.0	ksi				
r	8.2	radius of gyration				
K	1.0				Effective Length Factor	
Slenderness Ratio	58.3	<120	not-slender		considered for 14ft for unsupported length + 26ft depth to fixity; ratio for 40ft long length	AASHTO LFRD 6.9.3
D/t	43.4	<	64 ok	$\frac{D}{t} \leq 0.11\frac{E_{st}}{F_y}$	Local Buckling Check	AASHTO LFRD Table 6.9.4.2.1-1
Pe	3396	kips			Pe = Critical buckling load for a 40ft length	AASHTO LFRD Eq 6.9.4.1.2-1
Po	2013	kips			Max Axial Load based on Cross section AASHTO LFRD equation 6.9.4.1.2-1	
Poe	2214	kips	EXT Case			
Pe / Po	1.7					
Pe / Poe	1.5			If $\frac{P_e}{P_o} \geq 0.44$: $P_n = P_o 0.658^{\frac{P_o}{P_e}}$		
Pn	1571	kips				AASHTO LFRD Equation 6.9.4.1.1-1
Pne	1685	kips	EXT Case			
Φc factor	1.0		EXT Case			
Φc factor	0.7		STR Case			
Pr	1099	kips	STR Case	1. For members subjected to elastic forces:	Factored Axial Resistance	AASHTO LFRD Equation 6.9.2.1-1
Pre	1180	kips	EXT Case	$\frac{D}{t} \leq 0.22\frac{E}{F_y}$		
D/t	43.4	<	128		hence, plastic moment will yield pile	
D/t	43.4	<	87	2. For members subjected to plastic forces:: $\frac{D}{t} \leq 0.15\frac{E}{F_y}$	$M_n = \left(\frac{0.021E_{st}}{\frac{D}{t}} + F_y\right)S_y$ $M_n = M_p = F_y Z_y$	
Z	298.6	in3				
Mn	14932.2	kips-in			Nominal Moment resistance	
Φf factor	0.9		(0.9 used for STR case)		Flexural resistance factor	
Mr	13439.0	kips-in	STR Case		Factored Moment Resistance for steel casing only;	
Mre	16425.5	kips-in	EXT Case used to check the pile at 30ft length			
Vp	1207.8	kips			WSDOT 7.10.2-15 (considered steel casing only)	
Vr	1026.6	kips			(0.85) Factored Shear Resistance	AASHTO LFRD Eq 6.10.3.3-1
RCFST Mr=	18888	kips-in	use for STR case For first 25ft of pile		Moment resistance using RCFST composite section (nominal material properties) #9 rebars	
RCFST Mre=	23524	kips-in	use for EXT case For first 25ft of pile		Moment resistance using RCFST composite section (expected material properties) #9 rebars	
RCFST Mr=	21283	kips-in	use for STR case For first 25ft of pile		Moment resistance using RCFST composite section (nominal material properties) #11 rebars	
RCFST Mre=	26534	kips-in	use for EXT case For first 25ft of pile		Moment resistance using RCFST composite section (expected material properties) #11 rebars	

Abutment 1 Results: Case 1: EQ (Case 5)

NOTES:

Location 1: 0-10ft

Pu	351	kips		Factored Axial Load in Top 20ft
Pu / Pre	0.30		> 0.2	
Mux	1212	kips-in	Long	L-Pile Analysis
Muy	11400	kips-in	Trans	L-Pile Analysis
D/C	72%	ok		

If $\frac{P_u}{P_r} \geq 0.2$: $\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$ AASHTO LFRD Equation 6.9.2.2.1-2

Location 2: 10-15ft

Pu	351	kips		Factored Axial Load in Top 20ft
Pu / Pre	0.30		> 0.2	
Mux	10200	kips-in	Long	L-Pile Analysis
Muy	1900	kips-in	Trans	L-Pile Analysis
D/C	70%	ok		

If $\frac{P_u}{P_r} \geq 0.2$: $\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$ AASHTO LFRD Equation 6.9.2.2.1-2

Location 3: 15-25ft

Pu	351	kips		Factored Axial Load in Top 20ft
Pu / Pre	0.30		> 0.2	
Mux	14000	kips-in	Long	L-Pile Analysis
Muy	4400	kips-in	Trans	L-Pile Analysis
D/C	91%	ok		

If $\frac{P_u}{P_r} \geq 0.2$: $\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$ AASHTO LFRD Equation 6.9.2.2.1-2

Location 3: 30ft (Check using bare steel section only)

Pu	577	kips		Factored DL + DD
Pu / Pre	0.49		> 0.2	
Mux	300	kips-in	Long	L-Pile Analysis
Muy	2500	kips-in	Trans	L-Pile Analysis
D/C	64%	ok		

If $\frac{P_u}{P_r} \geq 0.2$: $\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$ AASHTO LFRD Equation 6.9.2.2.1-2

Hence, reinforcement can be terminated at 30ft

Location 4: 40ft

Pu	577	kips		Factored DL + DD
(Pu) / (Poe*0.5)	0.52		> 0.2	
Mux	0	kips-in		
Muy	0	kips-in		
D/C	52%	ok		Check for axial load only: Axial capacity = 0.5 x Fye x Ag; Fye = 1.1 x 50ksi

Location 5: 60ft

Pu	577	kips		Factored DL + DD
(Pu) / (Poe*0.7)	0	kips		
Mux	0.52		> 0.2	
Muy	0	kips-in		
	0	kips-in		
D/C	52%	ok		Check for axial load only: Axial capacity = 0.5 x Fye x Ag; Fye = 1.1 x 50ksi

Abutment 1 Results: STR Case

NOTES:

Location 1: 0-10ft

Pu	441	kips		Factored Axial Load in Top 20ft
Pu / Pr	0.40		> 0.2	
Mux	1236	kips-in	Long	L-Pile Analysis
Muy	500	kips-in	Trans	L-Pile Analysis
D/C	47%	ok		

If $\frac{P_u}{P_r} \geq 0.2$: $\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$ AASHTO LFRD Equation 6.9.2.2.1-2

Location 2: 10-15ft

Pu	441	kips		Factored Axial Load in Top 20ft
Pu / Pr	0.40		> 0.2	
Mux	5500	kips-in	Long	L-Pile Analysis
Muy	1000	kips-in	Trans	L-Pile Analysis
D/C	67%	ok		

If $\frac{P_u}{P_r} \geq 0.2$: $\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$ AASHTO LFRD Equation 6.9.2.2.1-2

Location 3: 15-25ft

Pu	441	kips		Factored Axial Load in Top 20ft
Pu / Pr	0.40		> 0.2	
Mux	5500	kips-in	Long	L-Pile Analysis
Muy	400	kips-in	Trans	L-Pile Analysis
D/C	65%	ok		

If $\frac{P_u}{P_r} \geq 0.2$: $\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$ AASHTO LFRD Equation 6.9.2.2.1-2

Location 4: 40ft

Pu	666	kips		Factored DL+DD
(Pu) / (Po*0.7)	0.47		> 0.2	
Mux	0	kips-in		
Muy	0	kips-in		
D/C	47%	ok		Check for axial load only: Axial capacity = 0.5 x Fy x Ag

Location 5: 60ft

Pu	666	kips		Factored DL+DD
(Pu) / (Po*0.7)	0.47		> 0.2	
Mux	0	kips-in		
Muy	0	kips-in		
D/C	47%	ok		Check for axial load only: Axial capacity = 0.5 x Fy x Ag

Abutment 2 Results: Case 1: EQ (Max of Case 5 and 6)

NOTES:

Location 1: 0-10ft

Pu	370	kips		Factored Axial Load in Top 20ft
Pu / Pre	0.31		> 0.2	
Mux	2000	kips-in	Long	L-Pile Analysis (30%L)
Muy	6000	kips-in	Trans	L-Pile Analysis (100%T)
D/C	62%	ok		If $\frac{P_u}{P_r} \geq 0.2$: $\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$ AASHTO LFRD Equation 6.9.2.2.1-2

Location 2: 10-15ft

Pu	370	kips		Factored Axial Load in Top 20ft
Pu / Pre	0.31		> 0.2	
Mux	4200	kips-in	Long	L-Pile Analysis (30%L)
Muy	1800	kips-in	Trans	L-Pile Analysis (100%T)
D/C	54%	ok		If $\frac{P_u}{P_r} \geq 0.2$: $\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$ AASHTO LFRD Equation 6.9.2.2.1-2

Location 3: 15-25ft

Pu	370	kips		Factored Axial Load in Top 20ft
Pu / Pre	0.31		> 0.2	
Mux	4200	kips-in	Long	L-Pile Analysis (30%L)
Muy	4200	kips-in	Trans	L-Pile Analysis (100%T)
D/C	63%	ok		If $\frac{P_u}{P_r} \geq 0.2$: $\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$ AASHTO LFRD Equation 6.9.2.2.1-2

Location 3: 30ft (Check using steel section only)

Pu	616	kips		Factored DL + DD
Pu / Pre	0.52		> 0.2	
Mux	0	kips-in	Long	L-Pile Analysis (30%L)
Muy	2000	kips-in	Trans	L-Pile Analysis (100%T)
D/C	63%	ok		If $\frac{P_u}{P_r} \geq 0.2$: $\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$ AASHTO LFRD Equation 6.9.2.2.1-2

Hence, reinforcement can be terminated at 30ft

Location 4: 40ft

Pu	616	kips		Factored DL + DD
(Pu) / (Poe*0.5)	0.56		> 0.2	
Mux	0	kips-in		
Muy	0	kips-in		
D/C	56%	ok		Check for axial load only: Axial capacity = 0.5 x Fye x Ag; Fye = 1.1 x 50ksi

Location 5: 60ft

Pu	616	kips		Factored DL + DD
(Pu) / (Poe*0.5)	0	kips		
Mux	0.56		> 0.2	
Muy	0	kips-in		
	0	kips-in		
D/C	56%	ok		Check for axial load only: Axial capacity = 0.5 x Fye x Ag; Fye = 1.1 x 50ksi

Abutment 2 Results: STR Case

NOTES:

Location 1: 0-10ft

Pu	509	kips		Factored Axial Load in Top 20ft
Pu / Pr	0.46		> 0.2	
Mux	2500	kips-in	Long	L-Pile Analysis
Muy	0	kips-in	Trans	L-Pile Analysis
D/C	58%	ok		If $\frac{P_u}{P_r} \geq 0.2$: $\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$ AASHTO LFRD Equation 6.9.2.2.1-2

Location 2: 10-15ft

Pu	509	kips		Factored Axial Load in Top 20ft
Pu / Pr	0.46		> 0.2	
Mux	3200	kips-in		L-Pile Analysis
Muy	250	kips-in		L-Pile Analysis
D/C	63%	ok		If $\frac{P_u}{P_r} \geq 0.2$: $\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$ AASHTO LFRD Equation 6.9.2.2.1-2

Location 3: 15-25ft

Pu	509	kips		Factored Axial Load in Top 20ft
Pu / Pr	0.46		> 0.2	
Mux	3500	kips-in		L-Pile Analysis
Muy	200	kips-in		L-Pile Analysis
D/C	64%	ok		If $\frac{P_u}{P_r} \geq 0.2$: $\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$ AASHTO LFRD Equation 6.9.2.2.1-2

Location 4: 40ft

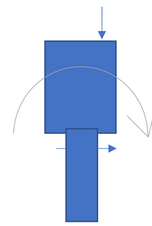
Pu	743	kips		Factored DL+DD
(Pu) / (Po*0.7)	0.47		> 0.2	
Mux	0	kips-in		
Muy	0	kips-in		
D/C	47%	ok		Check for axial load only: Axial capacity = 0.5 x Fy x Ag

Location 5: 60ft

Pu	743	kips		Factored DL+DD
(Pu) / (Po*0.7)	0.47		> 0.2	
Mux	0	kips-in		
Muy	0	kips-in		
D/C	47%	ok		Check for axial load only: Axial capacity = 0.5 x Fy x Ag

COWI	PROJECT:	I-405 R2B Final Design - BR 28W (112th Ave)	JOB NO.:	A207833
	SUBJECT:	Pile design check for jacking base		
	BY:	RSGR	DATE:	9/13/2021
	Check:	MEDN	DATE:	9/13/2021

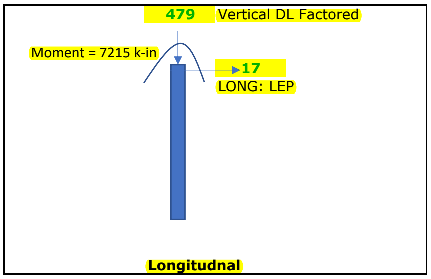
Max Girder Load DL	210	kips	
2 x DL=	420	kips	
Pile Cap weight per pile=	59	kips	
Jacking Load per pile, P=	420	kips	
eccentricity , e=	16	in	
Eccentric Moment, M(jacking)=	6720	kips-in	=420k x 16"
Active Lateral earth pressure from soil			
earth load factor=	1.5		
factored earth load, LEP=	17	kips	=11 x 1.5
Moment, M(LEP)=	495	kips-in	=2.5ft of moment arm
Total moment=	7215	kips-in	



Jack Load = 420 kips
eccentricity = 16"

Jacking Moment = 6720kip-in (Induced on top of pile)

Other lateral loads include earth pressure of 11 kips x 1.5 and induced moment of 675 kips-in

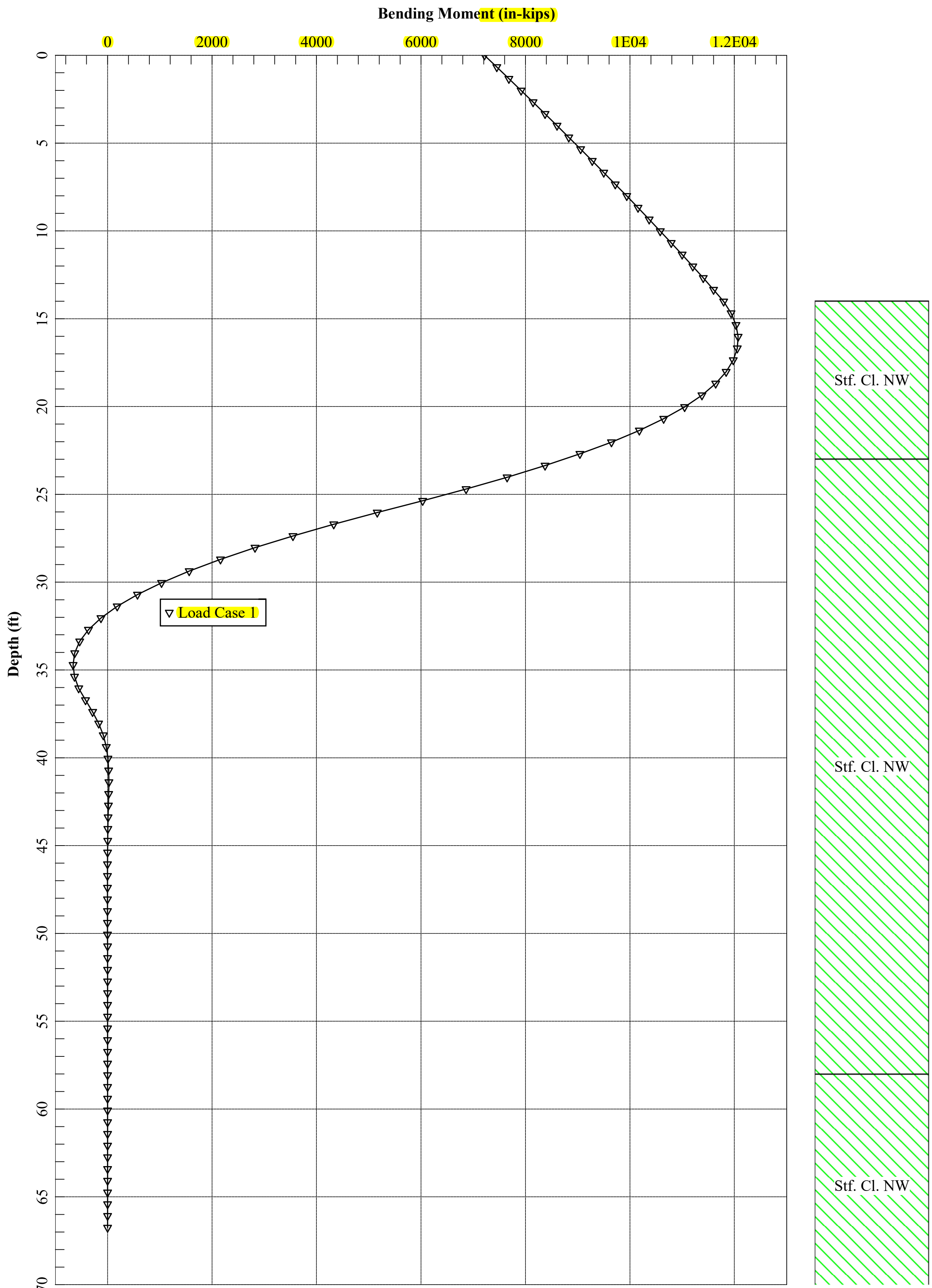


Factored Axial Resistance, Pr=	1099	kips	
Factored Flexural Resistance, Mr=	20987	kips-in	=composite section with #9 bundles (nominal) in abutment 2 (east)

Output from L-Pile

Pu=	479	kips	
Pu / Pr =	0.424022		
Mux =	12100	kips-in	=See L Pile Diagram
D/C	100%		=Pu/Pr + Mu/Mr

Moment diagram for jacking load case



BENDING RESISTANCE OF A CIRCULAR COMPOSITE COLUMN

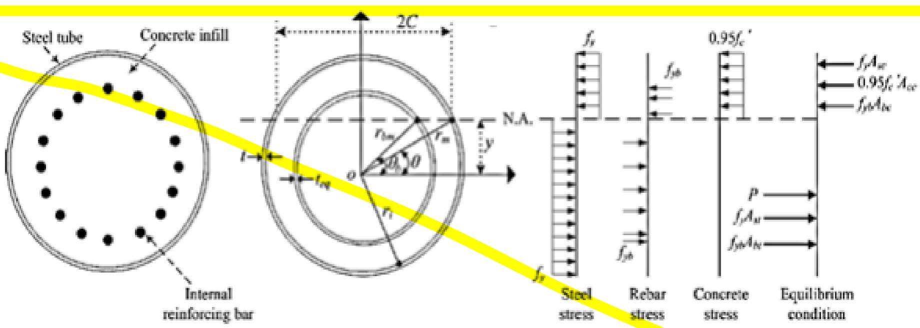
1.0 SECTION PARAMETERS

Diameter of pile	D=	24	in
Thickness	t=	0.625	in
Corossion outside	co=	0.1125	in
Corossion inside	ci=	0	in
Net Thickness	t,net=	0.5125	in
Outside Diameter after corrosion	D_O=	23.775	in
Inside Diameter after corrosion	D_i=	22.75	in
Area of steel after corrosion	Ag, net=	37.5	in^2
	E=	29000	ksi
	Fy=	50	ksi
	D (average)/t, net=	45.4	

$$\frac{D}{t} \leq 0.15\frac{E}{F_y}$$

45.4 < 87 No Local buckling

2.0 PLASTIC STRESS DISTRIBUTION MODEL



$$P_n(y) = \left(\left(\frac{\pi}{2} - \theta\right) r_i^2 - yc\right) * 0.95f'_c - 4\theta t r_m F_y - t_b r_{bm}\left(4\theta_b F_{yb} + (\pi - 2\theta_b)0.95f'_c\right) \quad (7.10.2\text{-}10)$$

$$M_n(y) = \left(c(r_i^2 - y^2) - \frac{c^3}{3}\right) * 0.95f'_c + 4ct\frac{r_m^2}{r_i}F_y + 4t_b r_{bm} c_b (F_{yb} - 0.95f'_c) \quad (7.10.2\text{-}11)$$

$$c_b = r_b cos\theta_b \quad (7.10.2\text{-}12)$$

$$\theta_b = sin^{-1}\left(\frac{y}{r_{bm}}\right) \quad (7.10.2\text{-}13)$$

$$t_b = \frac{nA_b}{2\pi r_{bm}} \quad (7.10.2\text{-}14)$$

Project:	i405 BR28 RC filled pile flexural capacity STR Case
Project #:	A207833 For #11 bundle
Calculation by:	RSGR

COWI

2.1 Assume theta, Θ

theta=

Assumed Theta	Θ=	0.350	rad	20.04727	deg
Assumed Theta b	Θb=	0.464	rad	26.58554	deg
Radius outside	r =	11.631	in		
Radius inside	r (i) =	11.375	in		
Radius centre of casing	rm=	11.503	in		
Radius to rebar cage	rbm =	8.811	in		
	t=	0.513	in		
	tb=	0.451	in	8 bundles of #11	
Neutral Axis	y=	3.943	in	=r x sin (theta)	
	c=	10.806	in	=r x cos (theta)	
	2c=	21.612	in		
	cb=	7.879	in	rb x cos (theta b)	

2.2 Check equilibirum

Area of concrete in compression	Acc=	116.307	in^2
Concrete strength	f'c=	5.000	ksi
Area of steel in compression	Asc=	14.556	in^2
Steel strength	Fy=	50.000	ksi
Area of steel rebar in compression	A'sc=	11.352	in^2
Steel strength	f'y=	60.000	ksi
Area of steel in tension	Ast=	22.898	in^2
Rebar steel strength	f'y=	60.000	ksi
Area of rebar in tension	A'rb=	13.608	in^2

Cc+Cs+C's-Ts-T's= 0.00

3.0 Nominal Flexural Capacity at no axial load

$$M_n(y) = \left(c(r_i^2 - y^2) - \frac{c^3}{3}\right) * 0.95f'_c + 4ct\frac{r_m^2}{r_i}F_y + 4t_b r_{bm} c_b (F_{yb} - 0.95f'_c)$$

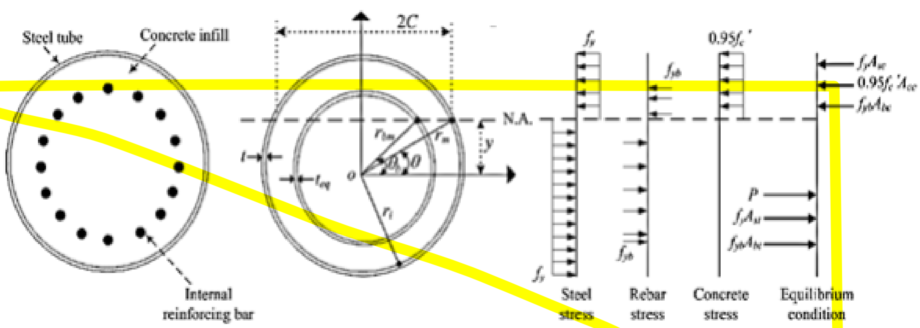
Mn=	23648	kips-in	nominal
Factored=	21283	kips-in	0.9 factor

COWI

BENDING RESISTANCE OF A CIRCULAR COMPOSITE COLUMN
Using expected material properties
1.0 SECTION PARAMETERS

Diameter of pile	D=	24	in		
Thickness	t=	0.625	in		
Corossion outside	co=	0.1125	in		
Corossion inside	ci=	0	in		
Net Thickness	t,net=	0.5125	in		
Outside Diameter after corrosion	D_O=	23.775	in		
Inside Diameter after corrosion	D_i=	22.75	in		
Area of steel after corrosion	Ag, net=	37.5	in^2		
	E=	29000	ksi		
	Fy=	55	ksi		=1.1 x 50 ksi
D (average)/t, net=		45.4		<	79.09091 No Local buckling $\frac{D}{t} \leq 0.15\frac{E}{F_y}$

2.0 PLASTIC STRESS DISTRIBUTION MODEL



$$P_n(y) = \left(\left(\frac{\pi}{2}-\theta\right) r_i^2 - yc\right) * 0.95f'_c - 4\theta t r_m F_y - t_b r_{bm}\left(4\theta_b F_{yb} + (\pi - 2\theta_b)0.95f'_c\right) \quad (7.10.2\text{-}10)$$

$$M_n(y) = \left(c(r_i^2 - y^2) - \frac{c^3}{3}\right) * 0.95f'_c + 4ct\frac{r_m^2}{r_i}F_y + 4t_b r_{bm} c_b (F_{yb} - 0.95f'_c) \quad (7.10.2\text{-}11)$$

$$c_b = r_b cos\theta_b \quad (7.10.2\text{-}12)$$

$$\theta_b = sin^{-1}\left(\frac{y}{r_{bm}}\right) \quad (7.10.2\text{-}13)$$

$$t_b = \frac{nA_b}{2\pi r_{bm}} \quad (7.10.2\text{-}14)$$

Project:	i405 BR28 RC filled pile flexural capacity EXT Case	
Project #:	A207833	For #11 bundle
Calculation by:	RSGR	

COWI

2.1 Assume theta, Θ

theta=

Assumed Theta	Θ=	0.373	rad	21.35876	deg
Assumed Theta b	Θb=	0.496	rad	28.39105	deg
Radius outside	r =	11.631	in		
Radius inside	r (i) =	11.375	in		
Radius centre of casing	rm=	11.503	in		
Radius to rebar cage	rbm =	8.811	in		
	t=	0.513	in		
	tb=	0.451	in	8 bundles of #11	
Neutral Axis	y=	4.190	in	=r x sin (theta)	
	c=	10.713	in	=r x cos (theta)	
	2c=	21.426	in		
	cb=	7.751	in	rb x cos (theta b)	

2.2 Check equilibirum

Area of concrete in compression	Acc=	111.124	in^2	
Concrete strength	f'c=	6.500	ksi	=1.3 x 5ksi
Area of steel in compression	Asc=	14.283	in^2	
Steel strength	Fy=	55.000	ksi	=1.1 x 50 ksi
Area of steel rebar in compression	A'sc=	11.029	in^2	
Steel rebar strength	f'y=	68.000	ksi	
Area of steel in tension	Ast=	23.171	in^2	
Rebar steel strength	f'y=	68.000	ksi	= 1.13 x 60 ksi
Area of rebar in tension	A'rb=	13.931	in^2	

Cc+Cs+C's-Ts-T's= 0.00 — Theta value results in equilibrium

3.0 Nominal Flexural Capacity at no axial load

$$M_n(y) = \left(c(r_i^2 - y^2) - \frac{c^3}{3}\right) * 0.95f'_c + 4ct\frac{r_m^2}{r_i}F_y + 4t_b r_{bm} c_b (F_{yb} - 0.95f'_c)$$

Mne= 26534 kips-in

Project:	i405 BR28 RC filled pile flexural capacity STR Case	
Project #:	A207833	For #9 bundles
Calculation by:	RSGR	

COWI

BENDING RESISTANCE OF A CIRCULAR COMPOSITE COLUMN

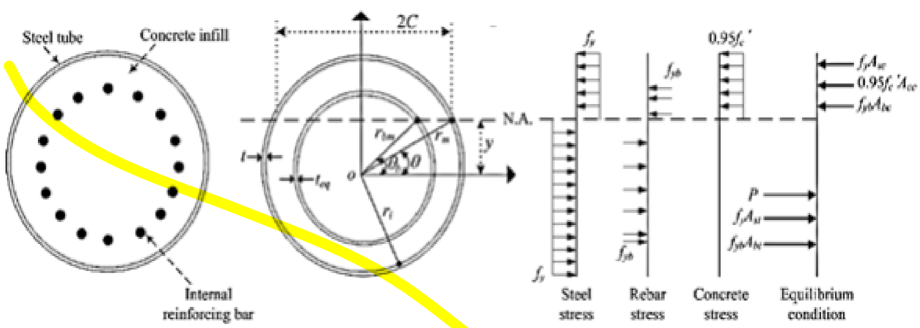
1.0 SECTION PARAMETERS

Diameter of pile	D=	24	in
Thickness	t=	0.625	in
Corossion outside	co=	0.1125	in
Corossion inside	ci=	0	in
Net Thickness	t,net=	0.5125	in
Outside Diameter after corrosion	D_O=	23.775	in
Inside Diameter after corrosion	D_i=	22.75	in
Area of steel after corrosion	Ag, net=	37.5	in^2
	E=	29000	ksi
	Fy=	50	ksi
	D (average)/t, net=	45.4	

$\frac{D}{t} \leq 0.15\frac{E}{F_y}$

45.4 < 87 No Local buckling

2.0 PLASTIC STRESS DISTRIBUTION MODEL



$$P_n(y) = \left(\left(\frac{\pi}{2} - \theta\right) r_i^2 - yc\right) * 0.95f'_c - 4\theta t r_m F_y - t_b r_{bm}\left(4\theta_b F_{yb} + (\pi - 2\theta_b)0.95f'_c\right) \quad (7.10.2\text{-}10)$$

$$M_n(y) = \left(c(r_i^2 - y^2) - \frac{c^3}{3}\right) * 0.95f'_c + 4ct\frac{r_m^2}{r_i}F_y + 4t_b r_{bm} c_b (F_{yb} - 0.95f'_c) \quad (7.10.2\text{-}11)$$

$$c_b = r_b \cos\theta_b \quad (7.10.2\text{-}12)$$

$$\theta_b = \sin^{-1}\left(\frac{y}{r_{bm}}\right) \quad (7.10.2\text{-}13)$$

$$t_b = \frac{nA_b}{2\pi r_{bm}} \quad (7.10.2\text{-}14)$$

COWI

2.1 Assume theta, Θ

theta=

Assumed Theta	Θ=	0.364	rad	20.87662	deg
Assumed Theta b	Θb=	0.484	rad	27.72565	deg
Radius outside	r =	11.631	in		
Radius inside	r (i) =	11.375	in		
Radius centre of casing	rm=	11.503	in		
Radius to rebar cage	rbm =	8.811	in		
	t=	0.513	in		
	tb=	0.289	in	8 bundles of #9	
Neutral Axis	y=	4.099	in	=r x sin (theta)	
	c=	10.748	in	=r x cos (theta)	
	2c=	21.496	in		
	cb=	7.799	in	rb x cos (theta b)	

2.2 Check equilibirum

Area of concrete in compression	Acc=	113.019	in^2
Concrete strength	f'c=	5.000	ksi
Area of steel in compression	Asc=	14.383	in^2
Steel strength	Fy=	50.000	ksi
Area of steel rebar in compression	A'sc=	7.146	in^2
Steel strength	f'y=	60.000	ksi
Area of steel in tension	Ast=	23.071	in^2
Rebar steel strength	f'y=	60.000	ksi
Area of rebar in tension	A'rb=	8.854	in^2

Cc+Cs+C's-Ts-T's= 0.00

3.0 Nominal Flexural Capacity at no axial load

$$M_n(y) = \left(c(r_i^2 - y^2) - \frac{c^3}{3}\right) * 0.95f'_c + 4ct\frac{r_m^2}{r_i}F_y + 4t_b r_{bm} c_b (F_{yb} - 0.95f'_c)$$

Mn=	20987	kips-in	nominal
Factored=	18888	kips-in	0.9 factor

COWI

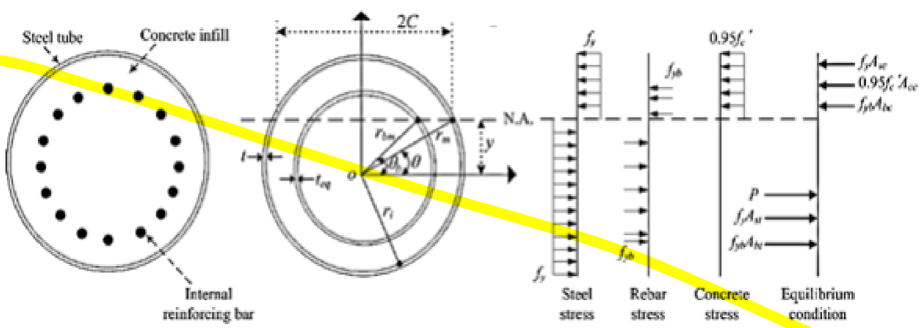
BENDING RESISTANCE OF A CIRCULAR COMPOSITE COLUMN

Using expected material properties

1.0 SECTION PARAMETERS

Diameter of pile	D=	24	in		
Thickness	t=	0.625	in		
Corossion outside	co=	0.1125	in		
Corossion inside	ci=	0	in		
Net Thickness	t,net=	0.5125	in		
Outside Diameter after corrosion	D_O=	23.775	in		
Inside Diameter after corrosion	D_i=	22.75	in		
Area of steel after corrosion	Ag, net=	37.5	in^2		
	E=	29000	ksi		
	Fy=	55	ksi	=1.1 x 50 ksi	
	D (average)/t, net=	45.4	<	79.09091	No Local buckling $\frac{D}{t} \leq 0.15\frac{E}{F_y}$

2.0 PLASTIC STRESS DISTRIBUTION MODEL



$$P_n(y) = \left(\left(\frac{\pi}{2} - \theta\right) r_i^2 - yc\right) * 0.95f'_c - 4\theta t r_m F_y - t_b r_{bm}\left(4\theta_b F_{yb} + (\pi - 2\theta_b)0.95f'_c\right) \quad (7.10.2\text{-}10)$$

$$M_n(y) = \left(c(r_i^2 - y^2) - \frac{c^3}{3}\right) * 0.95f'_c + 4ct\frac{r_m^2}{r_i}F_y + 4t_b r_{bm} c_b (F_{yb} - 0.95f'_c) \quad (7.10.2\text{-}11)$$

$$c_b = r_b cos\theta_b \quad (7.10.2\text{-}12)$$

$$\theta_b = sin^{-1}\left(\frac{y}{r_{bm}}\right) \quad (7.10.2\text{-}13)$$

$$t_b = \frac{nA_b}{2\pi r_{bm}} \quad (7.10.2\text{-}14)$$

COWI

2.1 Assume theta, Θ

theta=

Assumed Theta	Θ=	0.391	rad	22.39207	deg
Assumed Theta b	Θb=	0.521	rad	29.82392	deg
Radius outside	r =	11.631	in		
Radius inside	r (i) =	11.375	in		
Radius centre of casing	rm=	11.503	in		
Radius to rebar cage	rbm =	8.811	in		
	t=	0.513	in		
	tb=	0.289	in	8 bundles of #9	
Neutral Axis	y=	4.382	in	=r x sin (theta)	
	c=	10.636	in	=r x cos (theta)	
	2c=	21.272	in		
	cb=	7.644	in	rb x cos (theta b)	

2.2 Check equilibirum

Area of concrete in compression	Acc=	107.105	in^2	
Concrete strength	f'c=	6.500	ksi	=1.3 x 5ksi
Area of steel in compression	Asc=	14.068	in^2	
Steel strength	Fy=	55.000	ksi	=1.1 x 50 ksi
Area of steel rebar in compression	A'sc=	6.906	in^2	
Steel rebar strength	f'y=	68.000	ksi	
Area of steel in tension	Ast=	23.386	in^2	
Rebar steel strength	f'y=	68.000	ksi	= 1.13 x 60 ksi
Area of rebar in tension	A'rb=	9.094	in^2	

Cc+Cs+C's-Ts-T's= 0.00 Theta value results in equilibrium

3.0 Nominal Flexural Capacity at no axial load

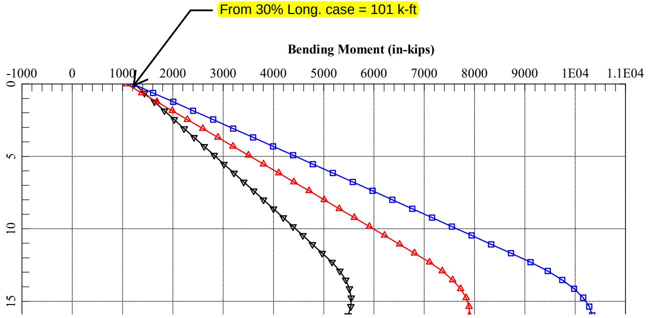
$$M_n(y) = \left(c({r_i}^2 - y^2) - \frac{c^3}{3}\right) * 0.95f'_c + 4ct\frac{{r_m}^2}{r_i}F_y + 4t_b r_{bm} c_b (F_{yb} - 0.95f'_c)$$

Mne= 23524 kips-in

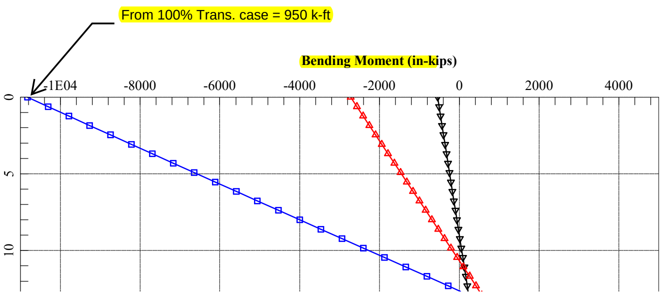
Check pile top connection at Abutment 1 (West)

CHECK PILE TOP CONNECTION

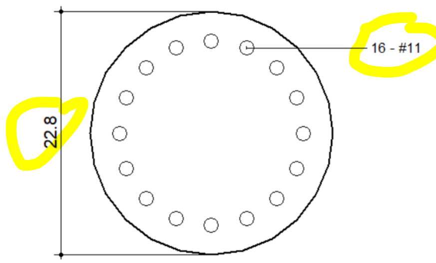
1. Longitudinal Moment



2. Transverse Moment



Geometric Properties	Gross Conc.	Trans (n=6.87)
Area (in^2)	401.8	548.3
Inertia (in^4)	12852.2	18299.1
y_t (in)	11.4	11.4
y_b (in)	11.4	11.4
S_t (in^3)	1129.9	1609.7
S_b (in^3)	1129.9	1607.8



Crack Spacing

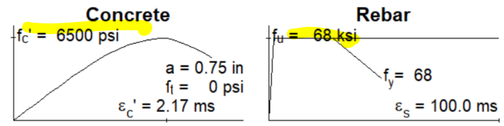
2 x dist + 0.1 d_b /ρ

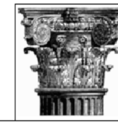
Loading (N,M,V + dN,dM,dV)

-220.0 , 0.0 , 0.0 + 0.0 , 1.0 , 0.0

All dimensions in inches

Clear cover to reinforcement = 2.02 in





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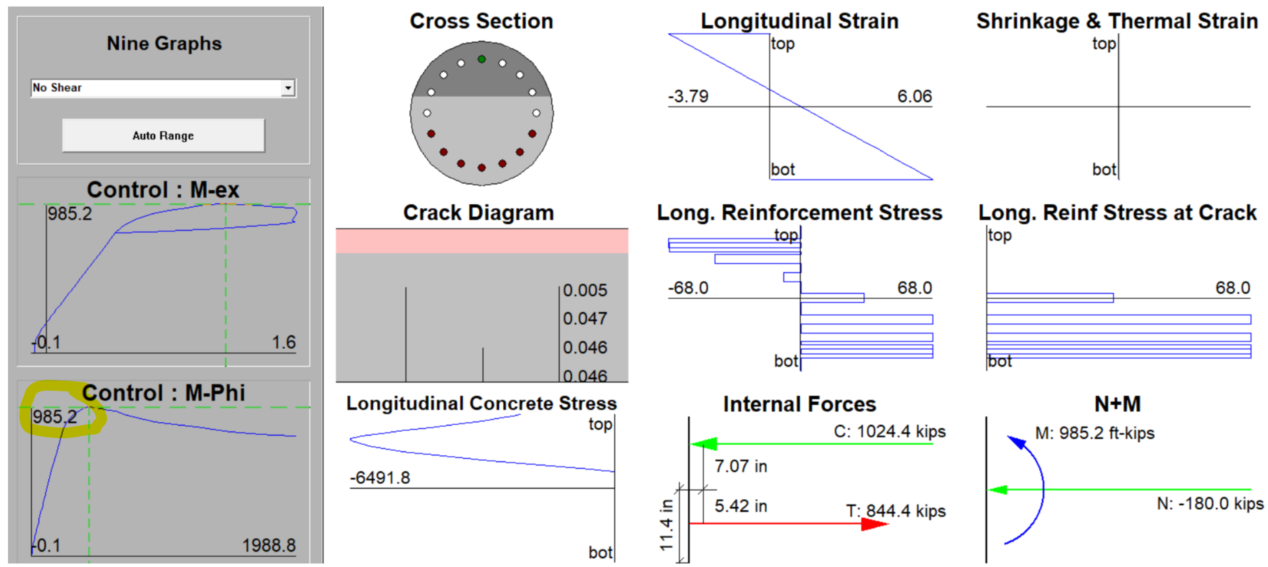


DEMAND = sqrt ($950^2 + 101^2$) = 955 k-ft

CAPACITY = 985 k-ft

D/C = 97%

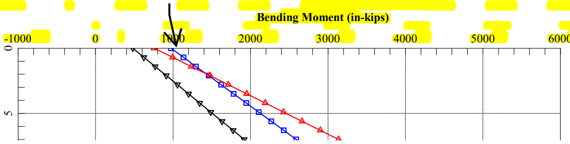
Check pile top connection at Abutment 2 (East)

Check Pile Top for Moment Capacity

1. Longitudnal Moment

From EXT Case
30% Long

82k-ft used to design pile
connection at top

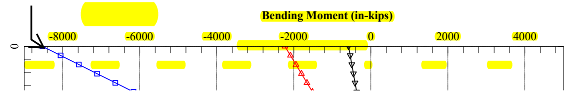


2. Transverse Moment

Transverse moment is checked from L Pile results.
In EXT case, the top moment is

From EXT Case
100% Trans

700k-ft



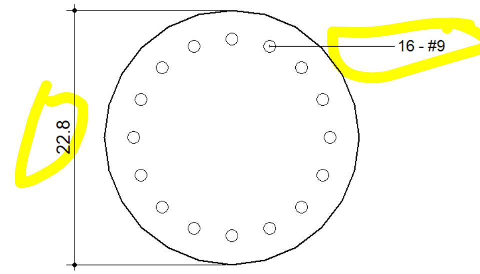
Design Moment Demand = sqrt (82 ^{2}+ 700^2) = 705k-ft

Moment capacity
(Using expected material properties) = 712 k-ft

Hence Capacity 712 k-ft > Demand 705k-ft

Geometric Properties

	Gross Conc.	Trans (n=6.87)
Area (in^2)	401.8	495.7
Inertia (in^4)	12852.2	16486.9
y_t (in)	11.4	11.4
y_b (in)	11.4	11.4
S_t (in^3)	1129.9	1450.0
S_b (in^3)	1129.9	1448.8

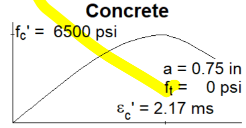


Crack Spacing
2 x dist + 0.1 d_b /ρ

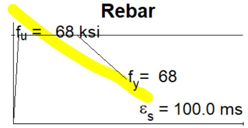
Loading (N,M,V + dN,dM,dV)
-152.0 , 0.0 , 0.0 + 0.0 , 1.0 , 0.0

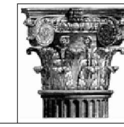
All dimensions in inches
Clear cover to reinforcement = 1.99 in

Concrete



Rebar





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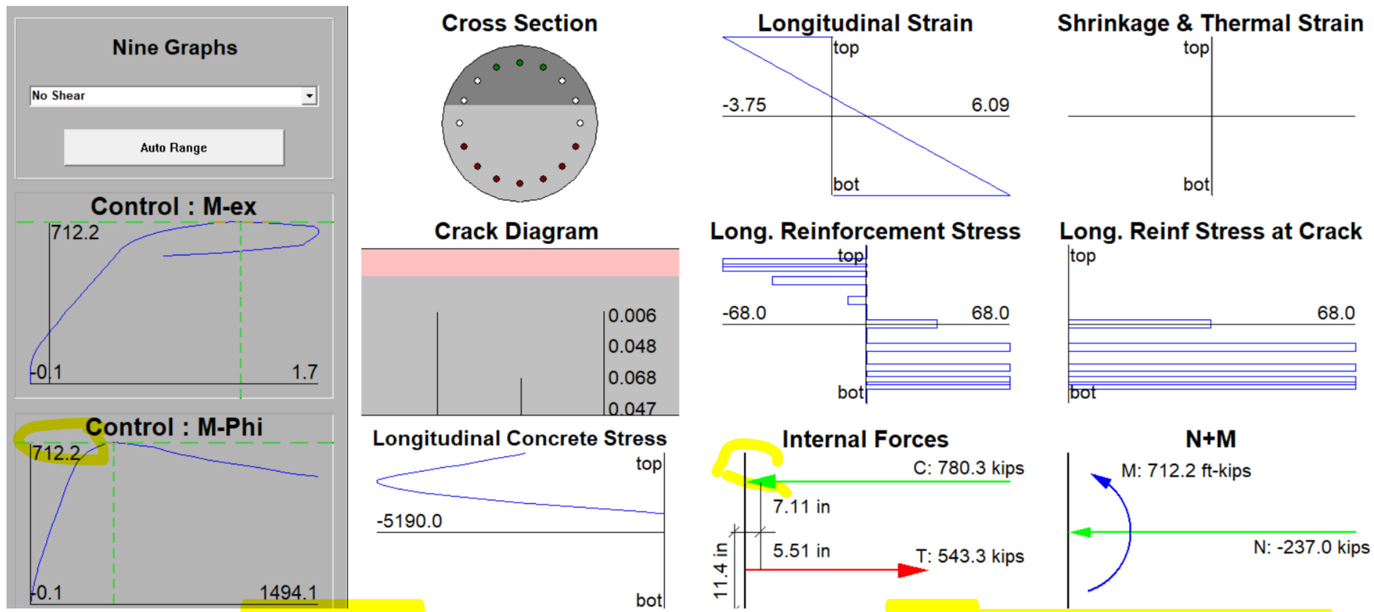

INDEX OF SHEETS

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BG28W-02	BRIDGE GENERAL NOTES
BG28W-03	CONSTRUCTION SEQUENCE
BG28W-04	FOUNDATION LAYOUT
BG28W-05	PILE DETAILS
BG28W-06	PIER 1 PLAN AND ELEVATION
BG28W-07	PIER 2 PLAN AND ELEVATION
BG28W-08	PIER DETAILS 1 OF 3
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BG28W-11	BEARING DETAILS
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BG28W-14	WF50G GIRDER DETAILS 1 OF 4
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GENERAL NOTES:

1. ALL MATERIALS AND WORKMANSHIP SHALL BE IN ACCORDANCE WITH THE REQUIREMENTS OF THE WASHINGTON STATE DEPARTMENT OF TRANSPORTATION "STANDARD SPECIFICATION FOR ROADS, BRIDGES, AND MUNICIPAL CONSTRUCTION", ENGLISH UNITS, DATED 2018 AND AMENDED JANUARY 7, 2019.

2. THIS STRUCTURE HAS BEEN DESIGNED IN ACCORDANCE WITH THE REQUIREMENTS OF THE "AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS", EIGHTH EDITION 2017. DEAD LOAD INCLUDES ADDITIONAL FUTURE WEARING SURFACE OF 35 POUNDS PER SQUARE FOOT ON THE ROADWAY SURFACE AND AN ALLOWANCE OF 110 POUNDS PER LINEAR FOOT FOR THE DUCTILE IRON WATER MAIN PIPE AND ATTACHMENTS. THE BRIDGE TRAFFIC BARRIERS HAVE BEEN DESIGNED IN ACCORDANCE WITH THE REQUIREMENTS FOR TEST LEVEL 4 (TL-4) RAILINGS.

3. THE SEISMIC DESIGN OF THIS STRUCTURE HAS BEEN COMPLETED IN ACCORDANCE WITH THE REQUIREMENTS OF THE "AASHTO GUIDE SPECIFICATIONS FOR LRFD SEISMIC BRIDGE DESIGN" SECOND EDITION 2011, WITH INTERIMS THROUGH 2015, USING SEISMIC DESIGN CATEGORY D, SITE CLASS D, AND THE FOLLOWING ACCELERATION PARAMETERS:

PARAMETER	SAFETY EVALUATION EARTHQUAKE (SEE)
PEAK GROUND ACCELERATION, PGA	0.430g
SITE-ADJUSTED PEAK GROUND ACCELERATION, A_S	0.503g
0.2 SEC SPECTRAL ACCELERATION, S_S	0.979g
1.0 SEC SPECTRAL ACCELERATION, S_1	0.283g

4. THE CONCRETE PILE PLUG SHALL BE CLASS 5000P.
THE CONCRETE FOR THE BRIDGE DECK SHALL BE CLASS 4000D.
THE CONCRETE IN APPROACH SLABS SHALL BE CLASS 4000A.
ALL OTHER CAST-IN-PLACE CONCRETE SHALL BE CLASS 4000.

5. REINFORCING BARS SHALL CONFORM TO ASTM A706 GRADE 60 UNLESS OTHERWISE NOTED.
UNLESS OTHERWISE NOTED, THE MINIMUM LAP SPLICE FOR BLACK REINFORCING BARS SHALL BE:

BAR SIZES:	#4	#5	#6	#7	#8	#9	#10	#11
SPLICE LENGTH (TOP BARS):	2'-0"	2'-7"	3'-1"	3'-7"	4'-1"	4'-7"	5'-2"	5-9"
SPLICE LENGTH (OTHERS):	2'-0"	2'-0"	2'-5"	2'-9"	3'-2"	3'-7"	4'-0"	4'-5"

THE ABOVE SPLICE LENGTHS ARE FOR CLASS B SPLICES WHERE λ_{rc} =0.4.
TOP BARS ARE HORIZONTAL BARS PLACED SUCH THAT MORE THAN 12" OF CONCRETE IS CAST BELOW REINFORCEMENT.

6. THE LOCATIONS OF ALL EXISTING UTILITIES WITHIN THE VICINITY OF THE STRUCTURE SHALL BE VERIFIED BY THE CONTRACTOR PRIOR TO EXCAVATION.

7. UNLESS OTHERWISE SHOWN ON THE PLANS, THE CONCRETE COVER MEASURED FROM THE FACE OF THE CONCRETE TO THE FACE OF ANY REINFORCEMENT BAR SHALL BE AS FOLLOWS:
   2½" TOP OF BRIDGE DECK
   1" BOTTOM OF BRIDGE DECK
   3" CAST AGAINST EARTH
   2" ALL OTHER LOCATIONS.

8. FALSEWORK SHALL BE CAREFULLY RELEASED TO PREVENT IMPACT OR UNDUE STRESS IN STRUCTURE.

9. THE BACKFILL BEHIND THE ABUTMENTS MAY BE PLACED PRIOR TO PLACEMENT OF THE SUPERSTRUCTURE, IN ACCORDANCE WITH STANDARD SPECIFICATION 2-09.3(1)E. BACKFILL AT THE PIER ABUTMENTS SHALL BE PLACED IN ACCORDANCE WITH STANDARD SPECIFICATION 2-03.3(14)I.

10. ALL EXPOSED CONCRETE CORNERS SHALL BE CHAMFERED ¾", UNLESS NOTED OTHERWISE.

11. FOR CONCRETE SLOPE PROTECTION, SEE WSDOT STD PLAN A-30.10.

12. EXISTING BRIDGE SHALL BE REMOVED TO 2'-0" MINIMUM BELOW FINISHED GRADE, AND AS REQUIRED TO ACCOMMODATE NEW CONSTRUCTION. AS-BUILT PLANS FOR THE EXISTING BRIDGE AND RETAINING WALLS ARE PROVIDED IN APPENDIX N2 OF THE CONFORMED RFP.

13. THE BRIDGE IS DESIGNED TO ACCOMMODATE FUTURE BEARING REPLACEMENT WITH THE FOLLOWING ASSUMPTIONS:
   - 1 JACK IS PLACED UNDER EACH GIRDER AT EACH PIER, AS SHOWN IN PIER LAYOUT SHEETS.
   - EACH JACK HAS A MINIMUM LIFTING CAPACITY OF 450 KIPS.
   - EACH JACK IS CENTERED ON A LOAD DISTRIBUTION PLATE WITH A MINIMUM AREA OF 140 IN² AND NOT PLACED CLOSER THAN 2" TO EDGE OF CONCRETE PILE CAPS.

LEGEND

A / 05 — IDENTIFIES SECTION, VIEW, OR DETAIL; TAKEN OR SHOWN ON PLAN REF NO BG28W-05

1 / - — TAKEN OR SHOWN ON CURRENT SHEET

▽ ← EPOXY COATED BAR

PJF = PREMOLDED JOINT FILLER

HOLD POINTS:

CONTRACTOR SHALL FOLLOW THE HOLD POINT PROCESS AS ESTABLISHED IN SECTION 2.28.5.4 OF THE CONFORMED RFP DOCUMENTS. MINIMUM HOLD POINTS FOR THIS BRIDGE ARE AS FOLLOWS:

- AFTER COMPLETION OF BRIDGE EXCAVATION AND EXISTING WALL DEMOLITION AND BEFORE THE START OF PILE DRIVING OPERATIONS.
- AFTER COMPLETION OF FIRST PILING DRIVEN FOR EACH ABUTMENT, INCLUDING TEST PILES
- AFTER INSTALLATION OF WALL 09.17L TO BOTTOM OF PILE CAP
- AFTER INSTALLATION OF GROUT PADS PRIOR TO SETTING BEARINGS
- AFTER GIRDER AND DIAPHRAGM PLACEMENT
- BEFORE CONCRETE PLACEMENT OF PILE PLUGS, PIER CAPS, DIAPHRAGMS, BRIDGE DECK, APPROACH SLABS, TRAFFIC BARRIERS, AND CURTAIN WALLS (WITH FORMWORK, INSERTS, AND REINFORCEMENT IN PLACE).

I-405 FILE NO. 9242 SHEET

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Design Mgr:	STEVE WOODRUFF	RELEASE FOR CONSTRUCTION RECORD		
Designed By:	M. BAUGHMAN			
Checked By:	M. DASTFAN			
Detailed By:	K. BUNGER			
Current Revision By:				
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REGION NO.	STATE	FED. AID. PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			

JOB NUMBER XL5467

MATTHEW D. BAUGHMAN · STATE OF WASHINGTON · 49664 · STRUCTURAL PROFESSIONAL ENGINEER

DATE — E.O.R. 1 STAMP BOX

DATE — E.O.R. 2 STAMP BOX

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Washington State Department of Transportation

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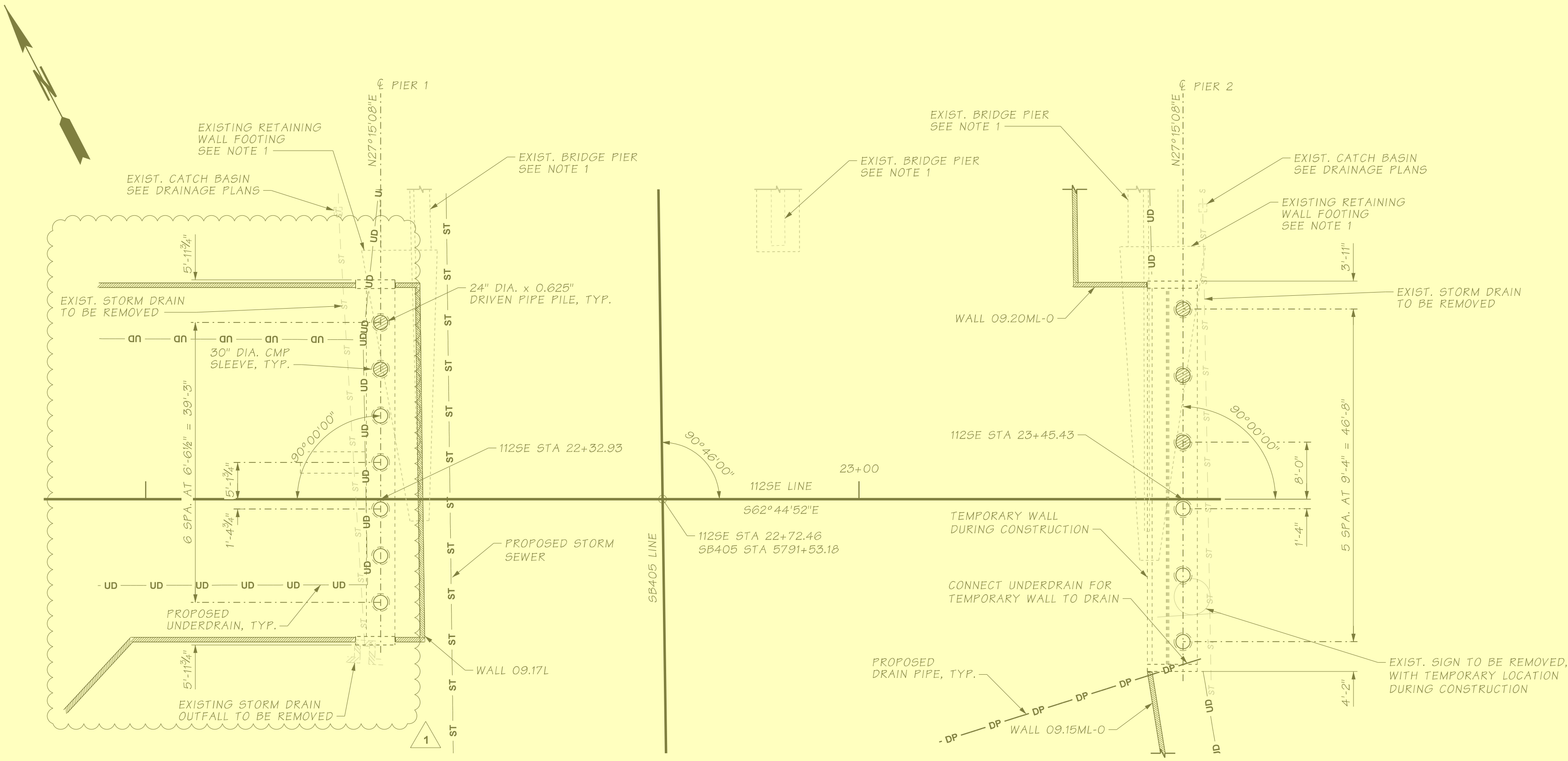
BRIDGE GENERAL NOTES

PLAN REF. NO. BG28W-02

SHEET OF SHEETS



FOUNDATION LAYOUT

PILES CONFLICT WITH EXISTING WALL FOOTING ACCORDING TO AS-BUILT DRAWINGS. PARTIAL DEMOLITION OF THE EXISTING FOOTING IS REQUIRED.

NOTES:

1. EXIST. BRIDGE PIER AND RETAINING WALLS TO BE REMOVED TO 2'-0" MIN. BELOW FINISHED GRADE.
2. FOR DRAINAGE DETAILS, SEE PLAN REF. NO. DR-34.
3. FOR RETAINING WALL DETAILS, SEE OTHER PACKAGE.

FILE NO. 9242 SHEET

I-405

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Designed By:	R. GARG			
Checked By:	M. DASTFAN			
Detailed By:	K. BUNGER			
Current Revision By:		NOC-57	09/21/2021	1
Date:	9/21/2021	RFC - BRIDGE 405/28W	06/28/2021	0
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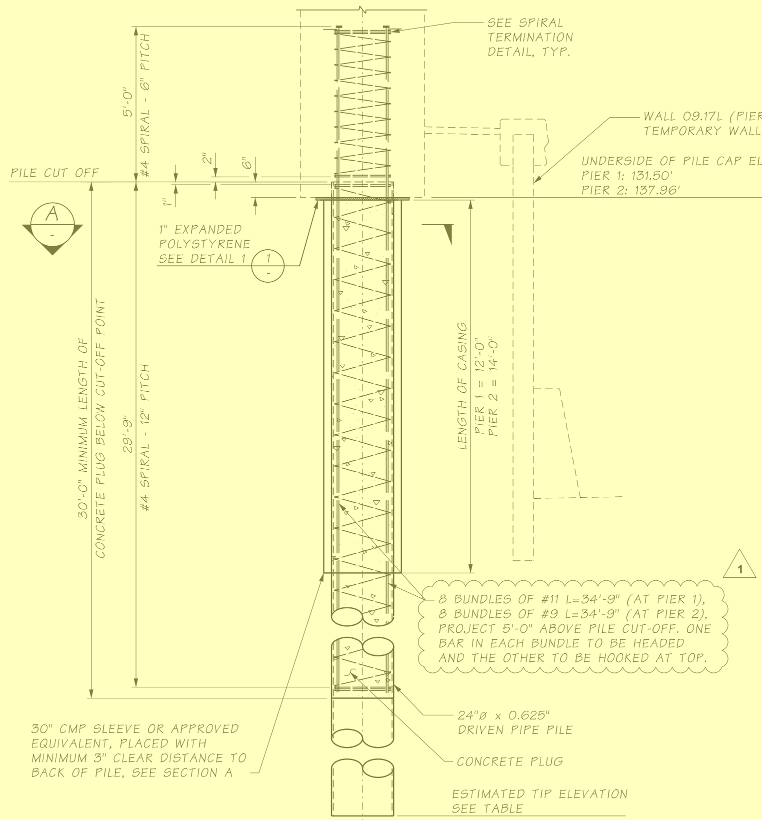
FOUNDATION PLAN

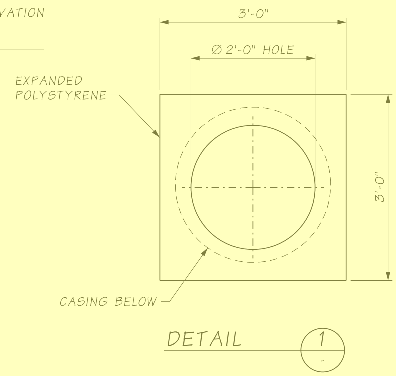
PLAN REF. NO. BG28W-04

SHEET OF SHEETS

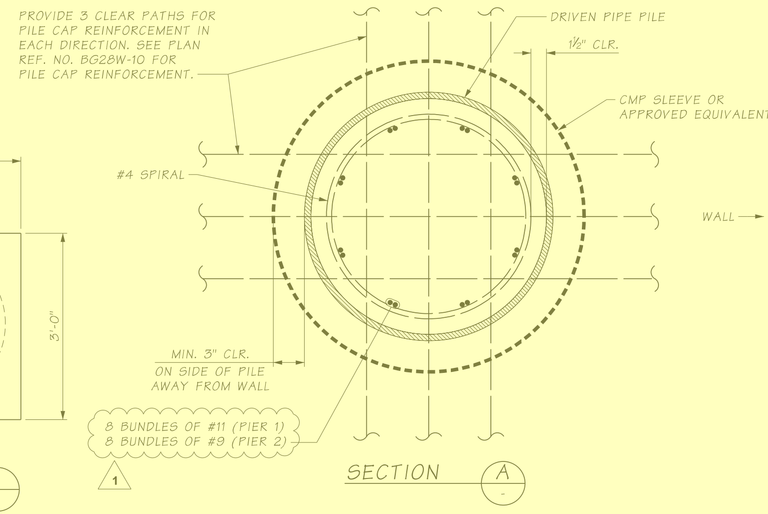


PILE ELEVATION

PIER 1 SHOWN, PIER 2 SIMILAR



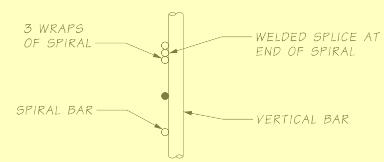
DETAIL 1



SECTION A

PIER	NUMBER OF PILES	PILE DIAMETER (D) IN.	STEEL THICKNESS (T) IN.	ESTIMATED TIP ELEV.	MIN. PILE TIP ELEV.	FACTORED DEMAND (KIPS)	MIN. BEARING CAPACITY (KIPS)	UNFACTORED DEAD LOAD (KIPS)	FACTORED DOWNDRAG FORCE (KIPS)
1	7	24	5/8	66	80	445	685	230	380
2	6	24	5/8	65	85	510	785	270	410

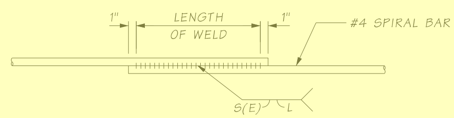
1. STEEL PIPE PILES SHALL COMPLY WITH ASTM A709, GRADE 50 (FY=50 KSI), API 5L X52, ASTM A588 GRADE 50, ASTM A572 GRADE 50, OR APPROVED EQUAL.
2. ALL PILES ARE ONLY UNDER COMPRESSION.
3. "FACTORED DEMAND" IS COMPRESSIVE DEMAND FROM STRENGTH 1 LOAD COMBINATION AT TOP OF PILE AND DOES NOT INCLUDE DOWNDRAG.
4. "MIN. BEARING CAPACITY" IS THE MINIMUM REQUIRED ULTIMATE CAPACITY ACHIEVED AT THE END OF PILE DRIVING. IT IS CALCULATED BY DIVIDING "FACTORED DEMAND" BY RESISTANCE FACTOR. RESISTANCE FACTOR OF 0.65 HAS BEEN USED FOR CALCULATION OF ULTIMATE PILE CAPACITY AND "ESTIMATED TIP ELEV.", ACCORDING TO "GEOTECHNICAL ENGINEERING REPORT: BRIDGE 28".
5. THE USE OF THE 0.65 COMPRESSION RESISTANCE FACTOR REQUIRES DEVELOPMENT OF DRIVING CRITERIA USING DYNAMIC TESTING AT THE BEGINNING OF THE PRODUCTION PILE DRIVING, QUALITY CONTROL BY DYNAMIC TESTING ON AT LEAST TWO PILES. GEOTECHNICAL ENGINEER TO DEFINE THE RE-STRIKE CRITERIA, IF NEEDED.
6. IF PILES REACH REFUSAL WITH PILE TIP HIGHER THAN THE SPECIFIED "MIN. PILE TIP ELEVATION", BRIDGE ENGINEER SHALL BE INFORMED TO ASSESS WHETHER PILES ARE ACCEPTABLE.
7. DOWNDRAG FORCES ON PILES HAVE NOT BEEN CONSIDERED FOR DETERMINING PILE LENGTHS. FACTORED DOWNDRAG FORCE IN COMBINATION WITH FACTORED DEAD LOAD HAS BEEN CONSIDERED FOR STRUCTURAL DESIGN OF PILES.
8. CORROSION RATES OF 0.0015 INCH-PER-YEAR WITHIN CASING, AND 0.001 INCH-PER-YEAR BELOW CASING HAS BEEN CONSIDERED FOR THE EXTERIOR FACE OF PILES ONLY.
9. STEEL PILE FABRICATION AND MATERIAL TESTING SHALL CONFORM TO SECTION 6-05.3(5).OPT1.GB6 OF THE GENERAL SPECIAL PROVISIONS IN APPENDIX B3 OF THE CONFORMED RFP.
10. STEEL PILE SPLICES SHALL CONFORM TO SECTION 6-05.3(6).OPT1.GB6 OF THE GENERAL SPECIAL PROVISIONS IN APPENDIX B3 OF THE CONFORMED RFP.
11. CMP SLEEVE SHALL BE 16 GAGE GALVANIZED STEEL DRAIN PIPE WITH 2⅔"x½" CORRUGATION IN ACCORDANCE WITH WSDOT STANDARD SPECIFICATION 9-05.1 OR APPROVED EQUIVALENT. CMP SLEEVE SHALL BE SECURED THROUGHOUT CONSTRUCTION TO MAINTAIN REQUIRED CLEARANCE TO PIPE PILES.



SPIRAL TERMINATION DETAIL

REQUIRED AT ALL LOCATIONS WHERE SPIRAL TERMINATES.

		WELD DIMENSIONS (IN)		
	BAR SIZE	S	E	L (LENGTH)
SPIRAL	#4	5/16	3/16	6



WELDED SPIRAL SPLICE DETAIL

WELDING SHALL MEET THE REQUIREMENTS OF STD. SPEC. 6-02.3(24)E. FOR WELD DIMENSIONS, SEE TABLE. NO WELDING OF SPIRALS IS PERMITTED AFTER THE COLUMN CAGE IS ASSEMBLED. SPIRAL SHALL BE FIELD OR SHOP WELDED INDEPENDENTLY AND THEN ASSEMBLED AROUND THE COLUMN LONGITUDINAL STEEL AFTER WELDING.

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Designed By:	R. GARG				
Checked By:	M. DASTFAN				
Detailed By:	K. BUNGER				
Current Revision By:		NDC-57	09/21/2021	1	
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JOB NUMBER XL5467

MATTHEW D. BAUGH, STATE OF WASHINGTON, 49664, STRUCTURAL PROFESSIONAL ENGINEER

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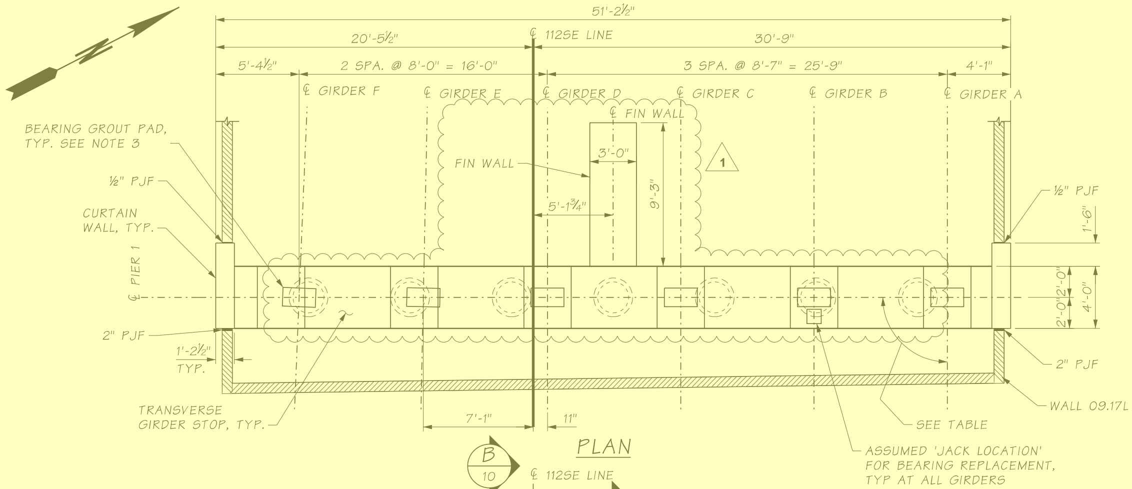
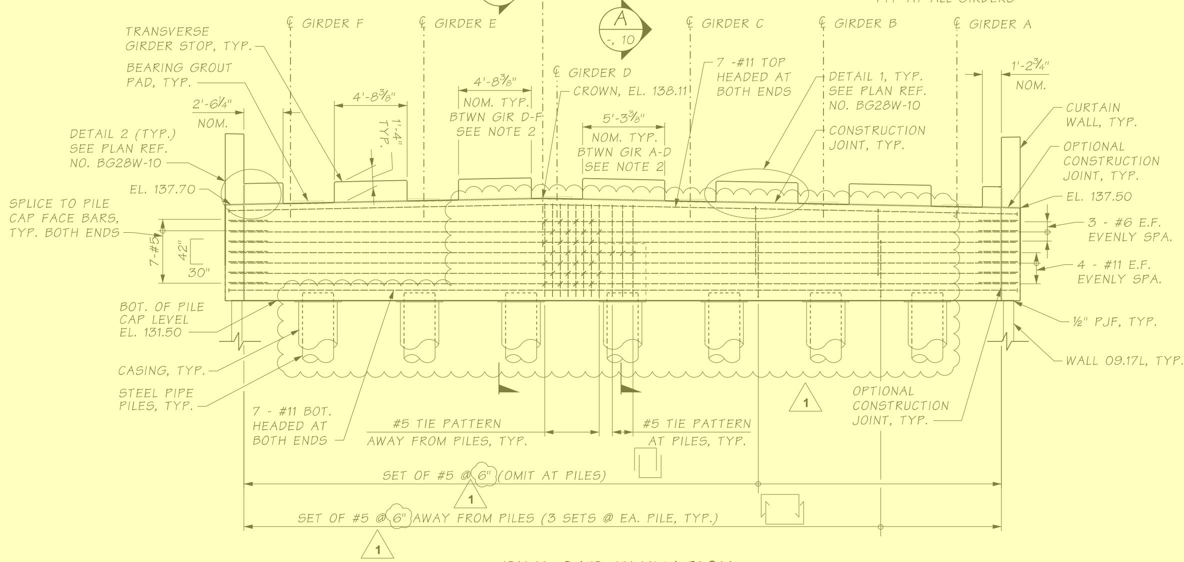
PILE DETAILS

PLAN REF. NO. BG28W-05

SHEET OF SHEETS



PLAN



PILE CAP ELEVATION

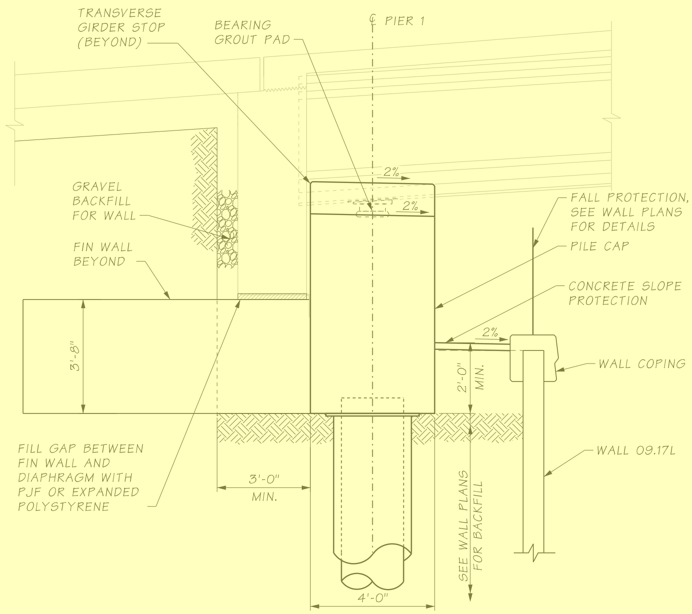
(LOOKING WEST)

DIMENSIONS AND ELEVATIONS ARE GIVEN ALONG ℄ PIER.
FIN WALL, GIRDER STOP AND CURTAIN WALL REINFORCEMENT NOT SHOWN, SEE PLAN REF. NO. BG28W-08 AND BG28W-10

NOTES:

1. FRONT SURFACE OF CONCRETE PILE CAP, GIRDER STOPS AND EXTERIOR FACE OF CURTAIN WALL SHALL RECEIVE "MT. ST. HELENS" PIGMENTED SEALER TO 1 FOOT BELOW FINISHED GRADE.
2. THE LENGTH OF TRANSVERSE GIRDER STOPS ARE NOMINAL. REFER TO PLAN REF. NO. BG28W-11 FOR DETAILS.
3. FOR DETAILS OF GROUT PAD RECESS, REFER TO PLAN REF. NO. BG28W-11.
4. FOR DETAILS OF WALL 09.17L, SEE OTHER PACKAGE.

TOP OF GROUT PAD	ELEVATION	ANGLE
GIRDER A	137.70	90°00'00"
GIRDER B	137.87	90°00'00"
GIRDER C	138.04	90°00'00"
GIRDER D	138.22	90°00'00"
GIRDER E	138.09	88°48'43"
GIRDER F	137.93	87°37'29"



SECTION A

FOR REINFORCEMENT DETAILS, SEE PLAN REF. NO. BG28W-10

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Design Mgr:	BRIAN BELL	RELEASE FOR CONSTRUCTION RECORD			
Designed By:	R. GARG				
Checked By:	M. DASTFAN				
Detailed By:	K. BUNGER				
Current Revision By:		NOC-57	09/21/2021	1	
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PIER 1 PLAN AND ELEVATION

PLAN REF. NO. BG28W-06

SHEET OF SHEETS

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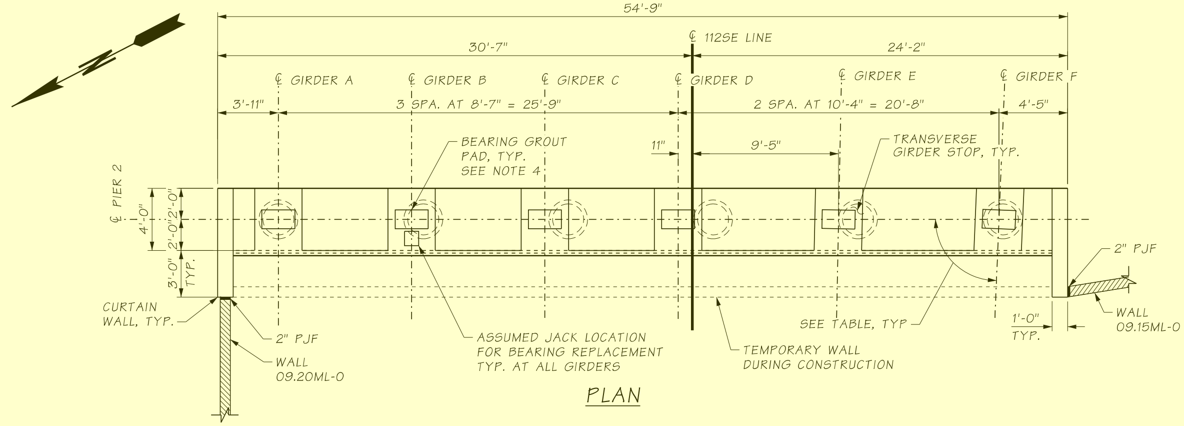

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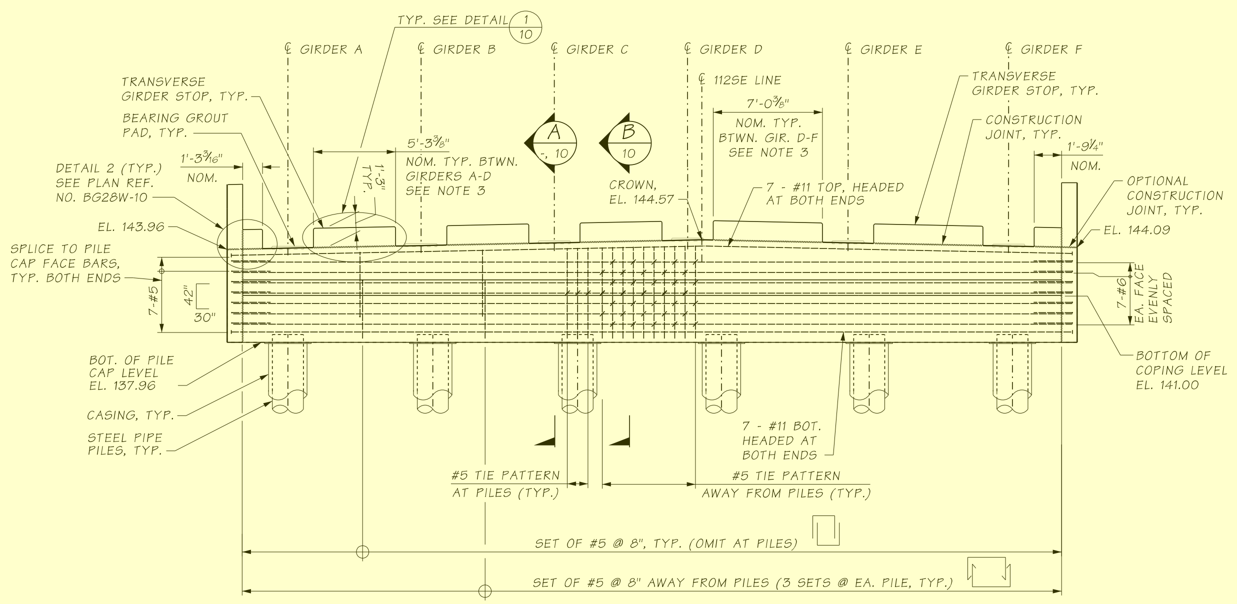


PILE CAP ELEVATION

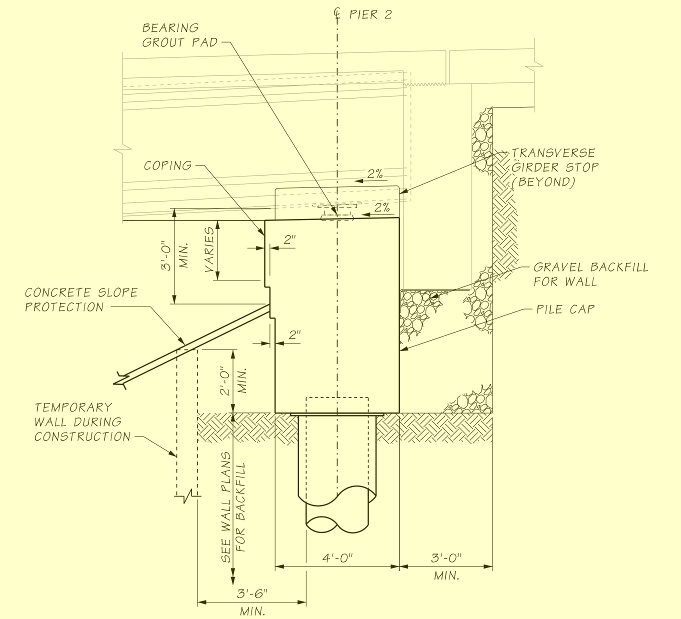
(LOOKING EAST)

DIMENSIONS AND ELEVATIONS ARE GIVEN ALONG ℄ PIER GIRDER STOP AND CURTAINWALL REINFORCEMENT NOT SHOWN, SEE PLAN REF. NO. BG28W-09 AND BG28W-10

NOTES:

1. FRONT SURFACE OF CONCRETE PILE CAP AND GIRDER STOPS SHALL RECEIVE "MT. ST. HELENS" PIGMENTED SEALER TO 1 FOOT BELOW FINISHED GRADE.
2. WALL COPING SHALL RECEIVE "MT. BAKER" PIGMENT SEALER.
3. THE LENGTH OF TRANSVERSE GIRDER STOPS ARE NOMINAL. REFER TO PLAN REF. NO. BG28W-11 FOR DETAILS.
4. FOR DETAILS OF GROUT PAD RECESS, REFER TO PLAN REF. NO. BG28W-11.
5. FOR DETAILS OF WALLS 09.15ML-0 AND 09.20ML-0, SEE OTHER PACKAGE.

TOP OF GROUT PAD	ELEVATION	ANGLE
GIRDER A	144.17	90°00'00"
GIRDER B	144.34	90°00'00"
GIRDER C	144.51	90°00'00"
GIRDER D	144.68	90°00'00"
GIRDER E	144.51	88°48'43"
GIRDER F	144.30	87°37'29"



SECTION A

FOR REINFORCEMENT DETAILS, SEE PLAN REF. NO. BG28W-10

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I-405 FILE NO. 9242 SHEET

Design Mgr:	STEVE WOODRUFF	RELEASE FOR CONSTRUCTION RECORD		
Designed By:	R. GARG			
Checked By:	M. DASTFAN			
Detailed By:	K. BUNGER			
Current Revision By:	P. CONRAD			
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REGION NO.	STATE	FED. AID. PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			

JOB NUMBER XL5467

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PIER 2 PLAN AND ELEVATION

PLAN REF. NO. BG28W-07

SHEET OF SHEETS

In Association with

Flatiron-Lane Joint Venture
Wood Environment & Infrastructure Solutions, Inc.
4020 Lake Washington Blvd NE, Suite 200
Kirkland, Washington 98033
T: 425-368-1000
www.woodplc.com

August 18, 2021

PS19-20316-0

Brian Bell
Interim Design Manager
Flatiron-Lane Joint Venture
400 Talbot Road South, Suite 400
Renton, WA 98055

Subject: **Bridge 28W – Fin Walls Addendum (BR28, Wall 09.17L, and EMB 2A-7)**
WSDOT I-405: Renton to Bellevue Widening and Express Toll Lanes Project
Renton, Washington

Dear Mr. Bell:

This addendum provides additional geotechnical design recommendations to the "released for use" (RFU) version of the Geotechnical Engineering Report: Bridge 28 (Hart Crowser, a division of Haley & Aldrich dated March 24, 2020) (submittal number 1169). This document is an addendum to that report.

The addendum includes geotechnical recommendations for the addition of a fin wall attached to the back of the BR 28W pile cap to resist transverse bridge loading under a seismic event. Note that the analyses presented in this letter are based on our understanding of the design changes to the "release for construction" (RFC) drawings dated June 28, 2021, as provided by the project Structural Engineer (Attachment 1) (Flatiron-Lane Joint Venture [FLJV] Submittal No. 1164). Based on discussions with the contractor and designers, we understand that a notice of design change (NDC) will be submitted to reflect the change from two fin walls to a single fin wall. A vicinity map, site plan, and subsurface profile for the west abutment are provided in Figures 1 through 4, respectively. All subsurface data (boring logs, groundwater measurements, laboratory data, etc.) are provided in appendices of the Bridge 28 report.

Structures Understanding

We understand one cast-in-place (CIP) fin wall (shear key) will be constructed behind the BR 28W-W abutment and will be constructed at the same time as the pile cap. The wall is currently designed to extend 9.3 feet behind the pile cap, extend approximately 3.67 feet above the bottom of pile cap, and be 3 feet wide. The fin wall will run parallel to the turnback portions of Wall 09.17L (bridge approach embankment walls). Therefore, no loading will occur from the fin wall to the front face of Wall 09.17L (portion of wall located below the bridge abutment), and will only load the turnback portions of the wall. We understand that one additional bridge pile has been added for the BR 28W-W abutment, and the piles have been shortened with the additional pile. We understand that preliminarily there are now 7 piles, spaced approximately 8 feet apart.

Soil Parameters

All soil parameters are consistent with those presented in the Bridge 28 report. We have assumed the abutment is backfilled with Washington State Department of Transportation (WSDOT) Gravel Borrow (9-03.14(1)), with a unit weight of 135 pounds per cubic foot (pcf) and a friction angle of 38 degrees.

Seismic Design

As detailed in the Bridge 28 report, we have assumed the walls are capable of sufficient movement under the WSDOT hazard level to allow using $k_h = 0.5\ k_{h0}$. If walls are not capable of such movement or wall movement is not desired, k_h is provided without reduction ($k_h = 1.0\ k_{h0}$). Table 1 provides the horizontal acceleration coefficients that were used to calculate the seismic earth pressure coefficients.

Table 1: Seismic Horizontal Acceleration Coefficient

Hazard	$k_h = 0.5\ k_{h0}$	$k_h = 1.0\ k_{h0}$
FEE	0.140 g	0.280 g
SEE/WSDOT	0.252 g	0.503 g

Lateral Earth Pressure Parameters

For mechanically stabilized earth (MSE) walls under seismic loading, the required external earth pressure diagrams are provided in Figure 5 per Section 11.10.7 of the American Association of State Highway and Transportation Officials (AASHTO) load resistance factored design (LRFD) Bridge Design Specifications. Internal stability pressures shall be calculated according to the Geotechnical Design Manual (GDM) and AASHTO as appropriate for the design method used. For gravity walls under seismic loading, the required earth pressure diagram is per AASHTO LRFD Section 11.6.5.1.

For the walls addressed in this report, we assume the walls are capable of sufficient movement under WSDOT seismic loading to allow for the use of $k_h = 0.5\ k_{h0}$, where k_{h0} is the seismic horizontal coefficient assuming zero wall displacement. Table 2 provides horizontal acceleration coefficients that were used to calculate the seismic earth pressure coefficients.

The lateral earth pressure parameters in Table 2 are for seismic conditions with flat ground behind the wall. Lateral earth pressure parameters were determined using the Mononobe-Okabe method. An interface friction angle of two-thirds of the soil internal friction angle (2/3*phi) was used to calculate the earth pressure parameters.

The earth pressure loads and resistances throughout this report do not include load or resistance factors. See AASHTO LRFD Section 11.5.7 for resistance factors for permanent walls.

Table 2: Fin Wall – Lateral Earth Pressure Coefficients (Flat Backslope and Vertical Wall Face)

Soil Type – Engineered Fill	K_h	Seismic Loading	
		K_{AE}	K_{PE}
WSDOT Gravel Borrow for Structural Earth Walls	0.252g	0.394	7.000

Fin Wall Resistance

The fin wall lateral resistance will be evaluated for both passive/punching shear resistance and sliding shear resistance cases. The lower resistance shall be considered the critical case. Based on discussions with the structural engineer, we understand the lateral pile resistances will be fully mobilized at approximately 1.2 inches of lateral movement. We discuss each analysis in the following sections. The analyses below use a fin wall size to provide the capacity required to carry the lateral load between the piles and the fin walls per the structural engineer. The sliding shear capacity controls, as discussed below, with a required resistance of at least 200 kips.

Passive/Punching Shear Resistance

The passive pressure wedge for the fin wall shall be consistent with GDM 15-5.2.4 Figure 15-4, which references Naval Facilities Engineering Command (NAVFAC) DM-7.02. To use the passive resistance as outlined in NAVFAC DM-7.02, the conditions of “ANCHOR WALL LEFT OF CC” in Figure 15-4 would need to be met. Based on our understanding of the current design, we do not meet the condition of $h1 \geq h2$ provided in NAVFAC DM-7.02. Therefore, the fin wall will be governed by the punching shear resistance, as indicated in the U.S. Army Corps of Engineers (USACE) EM 1110-2-2504.

As described in USACE, for anchors at large depth (i.e., where $h1 \leq h2$), the capacity of an anchor may be taken as the bearing capacity of a footing located at a depth equal to the midweight of the anchor. The bearing capacity of the footing shall be determined using AASHTO Section 10.6.3.1.2a, where the fin wall has a “bearing capacity” that is modified per Section 10.6.3.1.2b for a punching shear failure mechanism. In this scenario, the height of the fin wall shall be taken as the footing width and the length of the fin wall shall be taken as the footing length. These resistances include a reduction for the active wedge of the fin wall where the active wedge extends from the bottom of the fin wall to the final ground surface. This assumes that full passive pressure is mobilized, and that the bridge piles and fin wall have moved at least 1.4 inches (approximately 0.01H). The structural engineer will need to validate that the 1.4 inches of movement is realistic under the extreme loading condition. Using a fin wall length of 9.25 feet, the punching shear resistance is approximately 630 kips.

Sliding Shear Capacity

The sliding shear capacity shall be determined using AASHTO Guide Specifications for LRFD Seismic Bridge Design Section C6.4.3. Per Section C6.4.3, the sliding shear resistance is fully mobilized at about 0.5 inch or less of movement. This assumes the bridge piles, fin wall, and curtain walls behind the pile cap have moved about 0.5 inches. Using a fin wall length of 9.25 feet, the sliding shear resistance is approximately 200 kips. Based on our current understanding of the fin wall dimensions, the sliding shear capacity controls the fin wall lateral resistance.

The zone of influence for the sliding shear capacity (i.e., the length of soil being relied on to achieve the capacity) fin is 22 feet, measured from the near edge of the fin, as shown in Exhibit A, below. Due to the potential for settlement below the fin wall, we have not relied on sliding shear capacity below the fin walls for soil-on-fin contact.

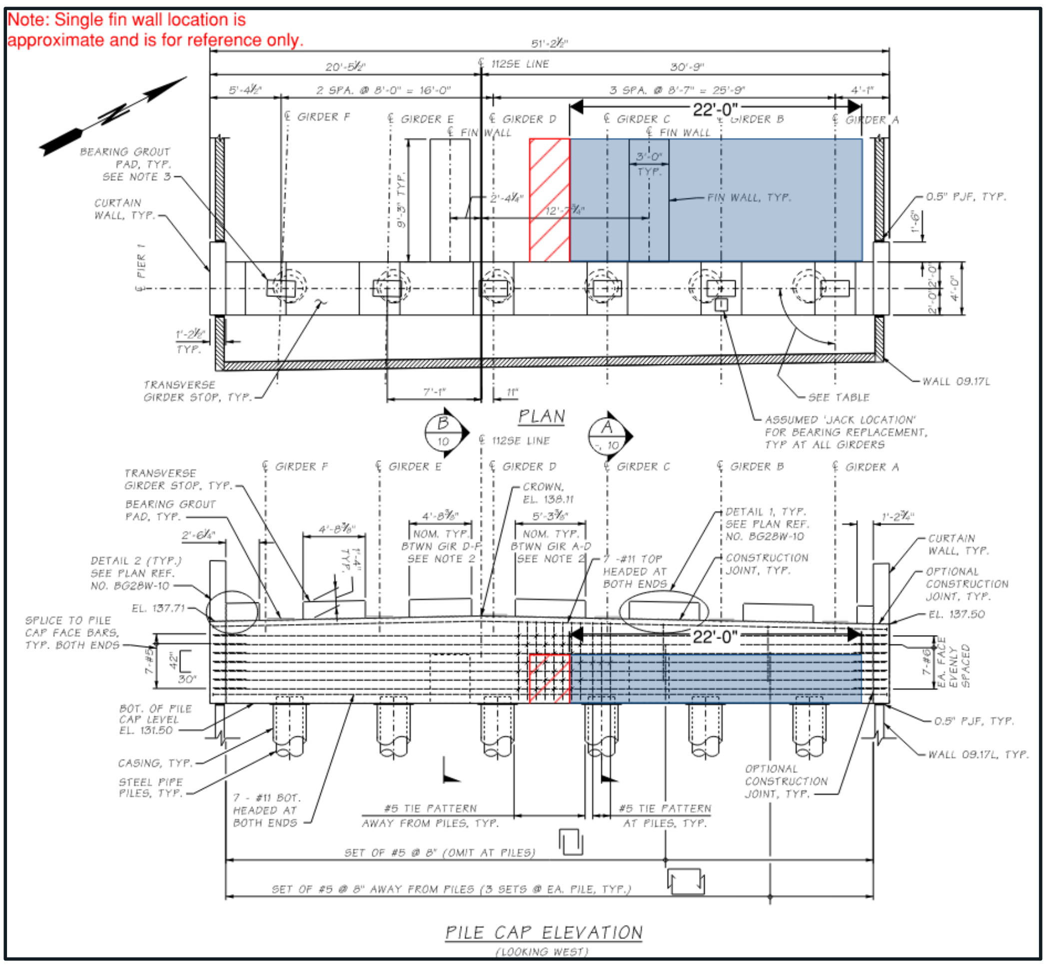


Exhibit A: Zone of influence for the sliding shear capacity

Fin Wall Conclusions

As described above, the governing loading case is the sliding shear capacity. We estimate that a 9.25-foot-long fin wall will provide approximately 200 kips of resistance, applied over the area of the fin. This results in an approximate pressure of 5.6 kips per square foot (ksf), assuming a 3.67-foot by 9.25-foot fin wall. The internal wall design shall include the fin wall load.

Note that due to this additional load, the MSE wall will not meet the design parameters and assumptions for preapproved walls per WSDOT GDM Section 15-A-3. Per Section 15-2, if a non-preapproved system is incorporated, the wall supplier shall completely design the wall prior to construction. Additionally, if a non-preapproved wall system is used, the wall design shall be submitted to The Bridge and Structures Office and the HQ Geotechnical Office. The design shall be in accordance with GDM Section 15-C.

MSE Lateral Sliding Resistance and Loading

The MSE sliding resistance for the 09.17L turnback wall was estimated based on AASHTO Figure 10.6.3.4-1 for footings resting on clay. The sliding resistance was calculated using the overburden stress and frictional resistance of the MSE wall. Per AASHTO Section 10.6.3.4 the strength is controlled by the lesser of the undrained shear strength or one-half the vertical effective stress. We included the frictional resistance based on our site-specific soil properties. The total horizontal force acting on the MSE wall consists of:

- The lateral load from the fin wall under extreme limit state loading (see Fin Wall Resistance and Loading Section of this report addendum);
- The lateral force due to the seismic active lateral earth pressure, P_{ae};
- The horizontal inertial force due to seismic loading, P_{ir}; and
- The traffic surcharge.

The horizontal inertial force imposed on the wall due to seismic loading was calculated using AASHTO Figure 11.10.7.1-1 (a). The minimum MSE reinforcement length, including the thickness of the wall facing, required to resist the horizontal loading is 21 feet. The extended reinforcement length is only required along the length of the fin wall. Outside of the fin wall extents, the minimum reinforcement length of 0.7H provides sufficient resistance against sliding.

MSE Overturning Resistance and Loading

The MSE overturning resistance and loading for the 09.17L turnback wall was estimated based on AASHTO Section 11.10.5.5 and The Federal Highway Administration (FHWA) NHI-10-024 Equation 4-15. We included the following loads as part of the overturning and resisting forces:

- Overturning – lateral load from the fin wall under extreme limit state loading (see Fin Wall Resistance and Loading Section of this report addendum);
- Overturning – lateral force due to the active lateral earth pressure under strength and extreme limit state loading;
- Overturning – traffic surcharge under strength and extreme limit state loading; and
- Resisting – self-weight of the reinforced section of the MSE wall assuming a reinforced width of 21 feet under strength and extreme limit state loading.

Based on our current understanding of the loads and dimensions of the MSE wall, the resultant reaction force is located within the middle two-thirds of the base width, per AASHTO Section 11.6.3.3.

Slope Stability Analyses

Global stability for the 09.17L turnback wall was calculated using the computer program SLIDE version 9.018 by Rocscience and critical rotational failure mechanisms were searched using both GLE/Morgenstern-Price and

Spencer limit equilibrium methods. Global stability and preliminary compound stability were analyzed for pseudostatic loading conditions only at one representative cross section of the walls. The following conditions are assumed:

- Per Section 2.13.4.1.1 of the request for proposal (RFP), a factored live load surcharge of 250 and 125 pounds per square foot (psf) has been applied for static and pseudostatic loading, respectively.
- Required factor of safety is 1.3 for static loading (resistance factor of 0.75).
- Required factor of safety is 1.1 for pseudostatic loading (resistance factor of 0.9).
- Pseudostatic lateral loading assumes the wall is sufficiently flexible to allow use of k_h=0.5 K_{h0}.
- Global and compound stability were analyzed using the Spencer and GLE/Morgenstern-Price method.
- The compound stability assessment was completed in accordance with Section 15-5.3.4 of the GDM.

It is our understanding that a specialty MSE wall designer will complete the internal stability analyses. For global stability runs, the MSE walls were modeled using an infinite strength material that extended back from the wall face 0.7H. We assumed an average wall height of 18 feet (16-foot exposed height with 2 feet of embedment) where the fin wall is located. Total wall heights and reinforced widths are presented in Table 3. The fin wall loading was applied at the fin wall face, equal to 5.6 ksf. This assumes a fin wall length of 9.3 feet based on the shear block resistance. The wall designer shall incorporate the 5.6 ksf load into their wall design. Outside of the fin wall extents, the fin wall load shall not be applied to wall 09.17L, and the design is at the discretion of FLJV and their wall designer.

For compound stability analyses, the MSE walls were modeled using 21-foot-long reinforcement strips at 2.46 feet vertical spacing. The strip tensile strength was 13.02 kips per lineal foot (klf) at 10 percent strip coverage over the width of the MSE wall. The interface angle between the reinforcements and wall backfill was 52 degrees at the top of wall, and decreased linearly to 27 degrees at the bottom reinforcement strip. Reinforcement strength and spacing was provided to us by the wall designer, Reinforced Earth Company (RECo). The reinforcement strip length was controlled by the MSE sliding analysis, as described above. Based on our analysis, the compound stability minimum factor of safety is not significantly affected by the strength of the lower 30 percent of the reinforcement, and appears to be primarily a function of the reinforcement strip length. Per WSDOT GDM Section 15-5.3.4, we reduced the tensile strength in the lower 30 percent of the reinforcement to 0.976 kips and 1.302 kips for static and pseudostatic loading, respectively. The reduced reinforcement strength analyses results maintained a factor of safety greater than the required minimums. See Figures A-5 through A-6 for results.

The results of the analysis are summarized in Table 3 and indicate that the overall stability of the wall meets the minimum required factors of safety, including the assumed conditions. We completed slope stability analyses for the static and pseudostatic cases, but have only included the fin wall loading under the pseudostatic case as the fin wall loading is only required under the extreme limit state. As shown in Table 3, all scenarios meet the required minimum factors of safety.

Table 3: Global Stability Results – Wall 09.17L

Analysis Section	Total Wall Height (feet)[a]	Reinforced Width (feet)	Scenario	Figure Number[b]	K_h[c,d]	Required Minimum Factor of Safety	Calculated Minimum Factor of Safety[e]
09.17L Turnback - Global	18	12.5	Static	A-1	--	1.3	3.6
		12.5	Pseudostatic	A-2	0.252	1.1	1.5
09.17L Turnback - Compound	18	21	Static	A-3	--	1.3	2.9
		21	Pseudostatic	A-4	0.252	1.1	1.4

Notes:

a. Total wall height measured from the top of the wall to the base of the wall footing or base of the embedded portion of the wall.
b. Figures presented in Appendix A.
c. Based on discussions with the structural designer (COWI), we understand the wall and bridge are capable of 1 to 2 inches of deformation during a seismic event and that such deformation is acceptable; therefore, $k_h = 0.5 k_{h0}$.
d. For global stability, minimum factors of safety were met without considering wave scattering effects. If needed, wave scattering effects can be incorporated in accordance with The National Cooperative Highway Research Program (NCHRP) 2011 for compound and internal stability.
e. The presented factor of safety is for the Spencer method.

Vertical Loading on Fin Walls

The vertical load from the soil column above the fin walls shall be incorporated as a permanent dead load for the bridge. Assume an unfactored soil unit weight of 135 pcf.

Settlement

Downdrag on Fin Wall

As described in Section 3.11.8 of AASHTO, downdrag occurs due to downward movement of the soil relative to the pile or shaft (or wall). This downward movement creates a drag load on the wall, which induces structural loads on the bridge pile and induces pile settlement. Assuming the fill around the fin wall settles more than the piles, we estimated the downdrag load on the fin walls using the methods outlined in FHWA-NHI-010-016. Assuming downdrag only occurs on three of the fin wall sides (north, south, and west sides), the downdrag load on the fin wall would be equal to 31 kips. In addition to the downdrag on the fin wall itself, the soil column above the fin wall will experience a drag load. The soil column drag load is equal to 27 kips, for a total downdrag load of 58 kips. Note that these values are nominal values, and load factors shall be applied in accordance with AASHTO LRFD Table 3.4.1-2. For the fin wall and soil column, the O'Neill and Reese Method for downdrag will apply.

Settlement Analyses

We previously completed settlement analyses for 09.17L using Settle3 (version 5.007), as detailed in the Bridge 28 report. The bridge loads were incorporated into the Settle3 models using the equivalent footing analysis approach discussed in detail in Section 7.6.4 of the Bridge 28 report. In our previous settlement analysis, we estimated a maximum total settlement over the 75-year design life of 1.5 inches and post-construction settlement of 1.0 inch. This included a 4.60 ksf load for the equivalent footing of the bridge. However, with the shortened piles, the neutral plane (i.e., the location at which the pile load acts) shifts upward. We originally assumed the neutral plane was applied at 21 feet below ground surface (bgs). To allow for flexibility for the structural pile design, we performed our analysis based on the minimum allowable pile tip elevation per the bridge structural plans (see Attachment 1). With the minimum pile length, the pile load is now applied at 23 feet bgs. Therefore, we have completed two updated settlement analyses, as described below.

Bridge and Wall Settlement with Fin Wall Load. This analysis is the same as our original analysis as described in Section 7.6.4 of the geotechnical report, but with an assumed pile tip elevation of 80 feet. Assuming the design minimum pile length is reached in the event of early refusal, the neutral plane would be located at approximately 23 feet bgs, or an approximate elevation of 99 feet. This analysis assumes that the fill below the fin wall settles more than the bridge and a gap forms between the top of fill and bottom of fin wall. This would result in the fin wall dead load and downdrag load being transferred to the pile as the fin wall and pile cap are rigidly connected. We applied an equivalent footing load of approximately 7.7 ksf, where 6.3 ksf is from the pile load, 0.6 ksf is from the fin load, including the weight of the soil column above the fin wall, and 0.8 ksf is from the downdrag on the fin wall and soil column.

Bridge and Wall Settlement with Fin Wall and EMB 2A-7 Loads. This analysis is the same as the analysis described above but includes the fill load from the 2A-7 embankments to the north and south of wall 09.17L.

The resulting settlement estimates using cone penetrometer test (CPT) based soil properties, shown in Tables 4 and 5 below, are still within the allowable settlement limits per Section 2.6.67 of the RFP and Table 15-3 of the GDM. Per the RFP, instrumentation would be required based on settlement results using soil properties from constant rate of strain consolidation (CRSCN) tests presented in Table 5. As discussed in the Bridge 28 design report, we are planning to instrument Wall 09.17L, which will be discussed further in a geotechnical instrumentation plan (GIP). Settlement figures for total settlement at 75 years and post-construction settlement at 75 years are presented in Appendix A (Figures A-7 through A-10). The values presented in Table 4 replace the values presented in Table 27 of the Bridge 28 report (FLJV Submittal No. 1169). Figure A-6 (post-construction settlement for CPT properties) replaces Figure FA-2 (Appendix F) of the Bridge 28 report and Figure A-8 (total settlement at 75 years for CPT properties) replaces Figure FA-3 (Appendix F) of the Bridge 28 report. Figure A-9 (post-construction settlement for CRSCN properties) replaces Figure FA-11 (Appendix F) of the Bridge 28 report and Figure A-10 (total settlement at 75 years for CRSCN properties) replaces Figure FA-12 (Appendix F) of the Bridge 28 report. Note that, based on discussion with the structural engineer, the vertical forces on the fin walls add to the axial loads on the piles, but they also cause a reduction of flexural demand on the pile.

Table 4: MSE Settlement Estimates using CPT Soil Properties

Load Case	Wall Station at which Maximum Settlement Occurs	Maximum Settlement at End of Construction[a] (inch)		Maximum Settlement at 75 Years (inch)		Post-Construction Settlement[b] (inch)	
		Original Estimate[a]	Estimate with Fin Wall	Original Estimate[a]	Estimate with Fin Wall	Original Estimate[a]	Estimate with Fin Wall
BR 28W-W + 09.17L + Fin Wall	1+60	0.50	0.62	1.50	1.86	1.00	1.24
BR 28W-W + 09.17L + Fin Wall + Emb 2A-7	1+60	0.50	0.79	1.50	2.06	1.00	1.27

Notes:

a. The original estimated settlement values are the estimated values from the Bridge 28 Geotechnical Design Report. That analysis included a lower equivalent footing load (i.e., deeper and fewer piles), no fin load, and no adjacent earthen embankment loads from Embankment 2A-7.

Table 5: MSE Settlement Estimates using CRSCN Soil Properties

Load Case	Wall Station at which Maximum Settlement Occurs	Maximum Settlement at End of Construction[a] (inch)		Maximum Settlement at 75 Years (inch)		Post-Construction Settlement[b] (inch)	
		Original Estimate[a]	Estimate with Fin Wall	Original Estimate[a]	Estimate with Fin Wall	Original Estimate[a]	Estimate with Fin Wall
BR 28W-W + 09.17L + Fin Wall	1+60	1.50	1.60	3.00	3.33	1.50	1.73
BR 28W-W + 09.17L + Fin Wall + Emb 2A-7	1+60	1.50	1.70	3.00	3.53	1.50	1.83

Notes:

a. The original estimated settlement values are the estimated values from the Bridge 28 Geotechnical Design Report. That analysis included a lower equivalent footing load (i.e., deeper and fewer piles), no fin load, and no adjacent earthen embankment loads from Embankment 2A-7.

Appendix B
Calculation Package

Seismic Active, Gravel Backfill for Walls

Mononobe-Okabe Method (M-O)

Pseudo-static analysis of seismic earth pressure on retaining structures

Job Name: job name
Job Number: J-#####-##

NOTES:

(1) Refer to Geotechnical Earthquake Engineering by Kramer before using
(2) Refer to Sections 11.5 & 11.6, 11.8.1.1, and Figure 11.11 a (Kramer)
Note: 1/3 to 1/2 peak ground surface accelerations are typically used in M-O equation (see Kramer Sect. 11.8.1.1, p. 494)
kv can be assumed =0 when using M-O method for typ. wall designs (Seed and Whitman ,1970; see Kramer p. 479)
(3) This method does not include a water table.
(4) This method is only recommended as a rough estimate for tiebacks.
(5) Check with hand calculations.
(6) Insert values into yellow areas.
(7) This method assumes a "Yielding" wall condition
(8) Paper: "Seismic Design and Behavior of Gravity Retaining Walls" by Robt. V. Whitman

Parameters	Symbol	Value	Units
Slope Inclination	β	0.000	radians
Horizontal Acceleration coef/g	k_h	0.252	
Vertical Acceleration coef/g	k_v	0	
Soil Friction Angle	ϕ	0.663	radians
Soil/Wall Friction Angle	δ	0.442	radians
Wall Angle (Batter) from Vertical	θ	0.000	radians
Unit Weight	γ	135	pcf
Height of Wall	H	1	feet

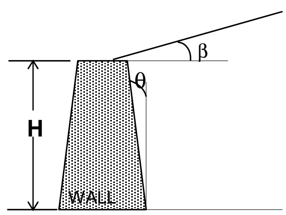
Conversion from degrees to radians:

Degrees	Radians
0	0.0000
38	0.6632
25.333	0.4422
0	0.0000

Reference

Slope	β
1.5H:1V	33.7
1.75H:1V	29.7
2H:1V	26.6
2.5H:1V	21.8
3H:1V	18.4
3.5H:1V	15.9
4H:1V	14.0
5H:1V	11.3
10H:1V	5.7

To be calculated	Symbol	Value
Coeff. of Active Earth Pressure	Ka	calculated
Active earth pressure resultant	Pa	calculated
Total Lateral Force	Pae	calculated
Dynamic Active Earth Pressure	Kae	calculated
Total thrust acts at this height:	h	calculated
Critical Failure Surface from Horz.	α_{ea}	calculated



Active Earth Pressure Calculations

Active Earth Pressure Coefficient	Ka	=	0.217	
Active thrust static component	Pa	=	15	pounds/foot
ArcTan(kh/(1-kv))=	ψ	=	0.2469	radians = 14.1 degrees
Dynamic active earth press. coef.	Kae	=	0.394	
Total Active Thrust	Pae	=	27	pounds/foot
Active thrust dynamic component	ΔPae	=	12	pounds/foot
Total Active Thrust acts at:	h	=	0.5	feet
Overturning moment about base	Mo	=	11	ft-lb/ft

Static Equivalent Active Fluid Unit Weight = 29 pcf
Dynamic Equivalent Active Fluid Unit Weight = 53 pcf
Dynamic Uniform Lateral Surcharge = 12 psf or = 12.0H

Note: The dynamic portion of the equivalent fluid unit weight is typically applied as a rectangular distribution rather than a triangular distribution.

The above calculations	C1E		1.85	
correspond to a critical	C2E		3.23	
failure surface angle of	α_{EA}	=	0.828	radians = 47 degrees above horizontal

Static conditions produce a critical				
failure surface angle of	α_S	=	1.117	radians = 64 degrees above horizontal

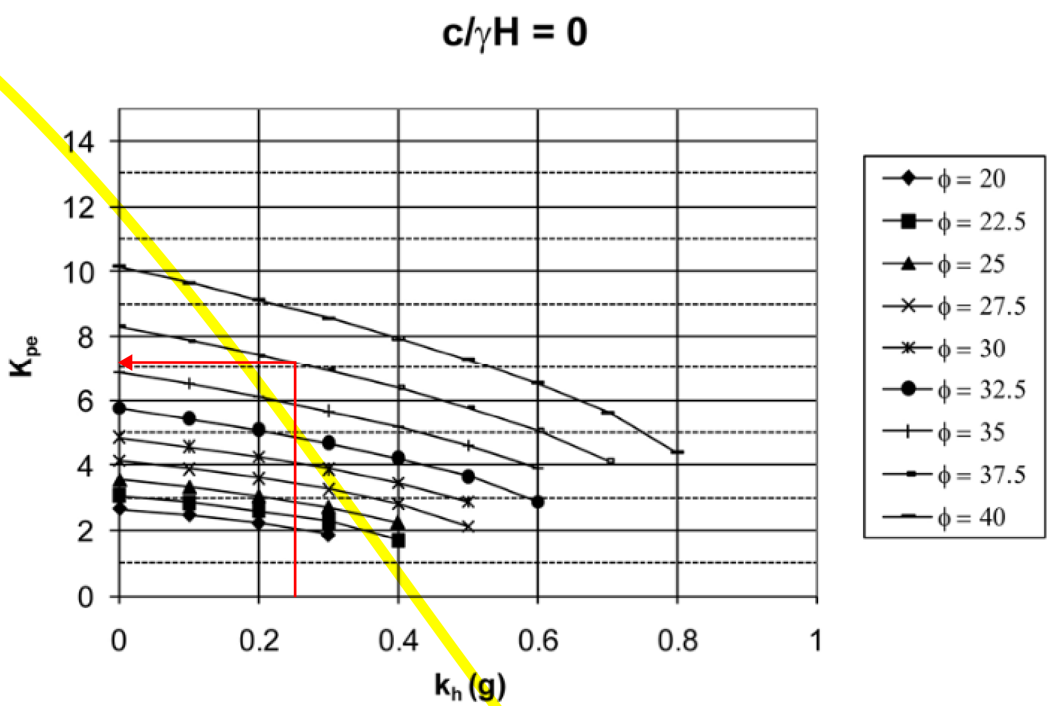
Passive Earth Pressure Calculations

Passive Earth Pressure Coefficien	Kp	=	14.22074	(Coulomb)
Passive thrust static component	Pp	=	960	pounds/foot
Dynamic passive earth press. coef	Kpe	=	11.21	
Total Passive Thrust	Ppe	=	757	pounds/foot
Passive thrust dynamic componen	ΔPpe	=	-203	pounds/foot

Static Equivalent Passive Fluid Unit Weight = 1920 pcf
Dynamic Equivalent Passive Fluid Unit Weight = 1513 pcf

The above calculations	C3E		1.85	
correspond to a critical	C4E		3.23	
failure surface angle of	α_{PE}	=	0.35	radians = 20 degrees above horizontal

Seismic Passive, Gravel Backfill for Walls



Kh = 0.252
Phi = 38 deg
c = 0 psf
Gamma = 135 pcf
H = 4 ft

Phi = 37.5, Kpe = 7.00

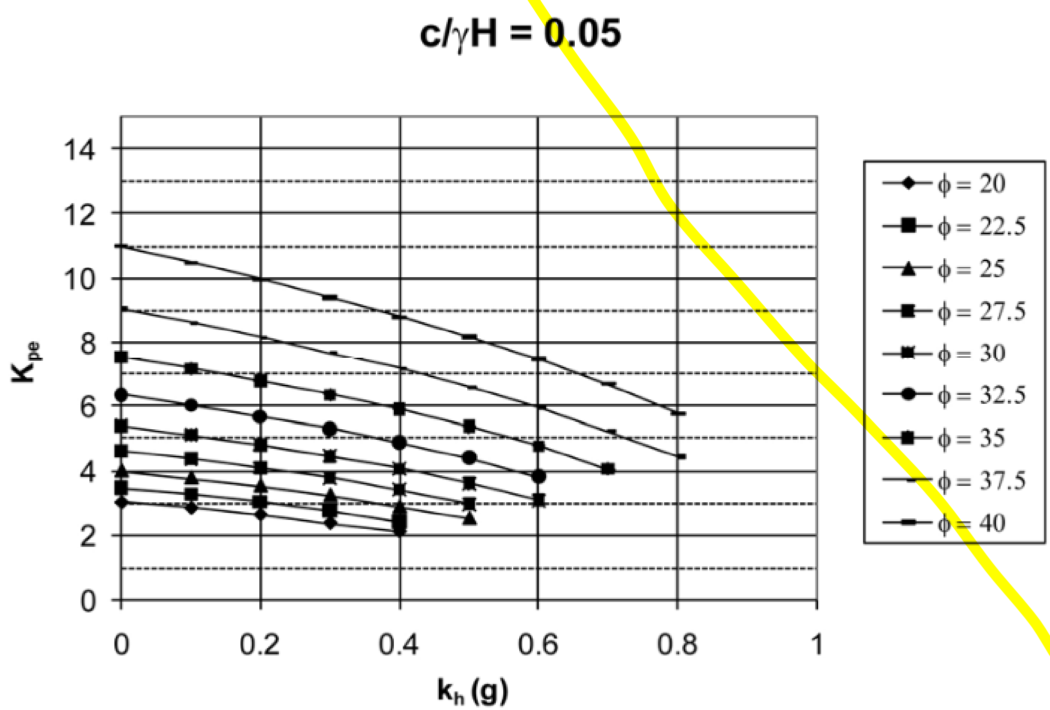


Figure A11.4-2—Seismic Passive Earth Pressure Coefficient Based on Log Spiral Procedure for $c/\gamma H = 0$ and 0.05 (c = soil cohesion, γ = soil unit weight, and H = height or depth of wall over which the passive resistance acts)

Note: $k_h = A_s = k_{ho}$ for wall heights greater than 20.0 ft.

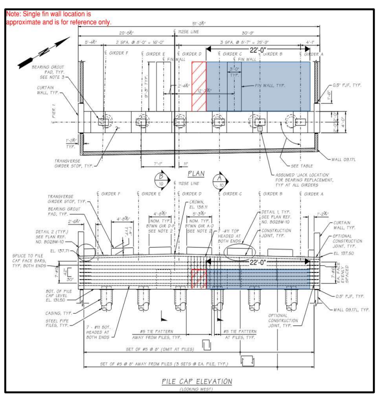
Fin Wall Shear Resistance

Sliding Shear

	Dimension	Unit	Center Fin
INPUTS	Height of MSE wall	ft	18
	Width of fin into page	ft	9.25
	Height from ground surface to top of fin	ft	7.67
	Height from ground surface to bottom of fin	ft	11.5
	Length of soil in front of fin	ft	22
	Height of fin	ft	3.83
	Thickness of fin	ft	3
	Gamma	pcf	135
	Phi	deg	38
	Ka	--	0.217
	Kae	--	0.394
	Kh	g	0.252
	Nq	--	--
	dq	--	--
	iq	--	--
	Cwq	--	--
	Ngamma	--	--
	igamma	--	--
	Cwgamma	--	--
OUTPUTS	Shear at bottom of soil in front of fin	kips	247
	Shear at side of soil in front of fin	kips	18
	Shear at side of fin	kips	3
	Active wedge (fin)	kips	4
	Active wedge (MSE)	kips	33
	PIR	kips	33
	Total shear	kips	199
	Pressure	ksf	5.6
	Sum shear	kips	199

Punching Shear

	Dimension	Unit	Center Fin
INPUTS	Height of MSE wall	ft	18.00
	Width of fin into page	ft	9.25
	Height from ground surface to top of fin	ft	7.67
	Height from ground surface to bottom of fin	ft	11.5
	Length of soil in front of fin	ft	22
	Height of fin	ft	3.83
	Thickness of fin	ft	3
	Gamma	pcf	135
	Phi	deg	38
	Ka	--	0.217
	Kae	--	0.394
	Kh	g	0.252
	Nq	--	13.20
	dq	--	1.00
	iq	--	1.00
	Cwq	--	1.00
	Ngamma	--	14.50
	igamma	--	1.00
	Cwgamma	--	1.00
OUTPUTS	Phi (shear)	deg	27.63
	Df	ft	7.67
	sq	--	1.22
	sgamma	--	0.83
	Nqm	--	16.06
	Ngammam	--	12.10
	qn	ksf	19.76
	Active wedge (fin)	kips	4
	Active wedge (MSE)	kips	33
	PIR	kips	33
	qn	kips	631
	Sum qn	kips	631

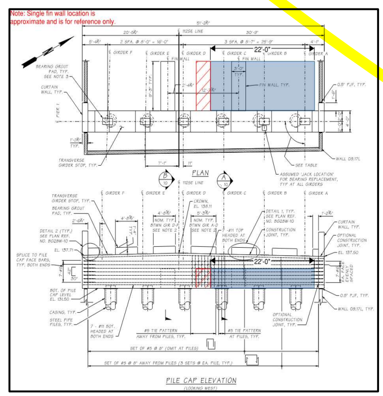


C6.4.3

Lateral capacity of the pile cap should include the passive pressure mobilized at the face of the cap and the interface shear resistance developed along each side of the cap. Procedures used to estimate the passive pressure at the face of the cap can normally involve static passive pressure equations and charts given in Section 3 of the *AASHTO LRFD Bridge Design Specifications.* Wall friction of two-thirds of the friction angle should be used in this determination. The amount of displacement to mobilize the passive pressure should follow guidance given in Section 10 of the *AASHTO LRFD Bridge Design Specifications*.

The shear along the side of the cap can be estimated using the effective pressure at the mid-height of the cap thickness (σ_v'), a lateral stress factor (K_o) of 0.5, and the friction angle (ϕ) of the backfill material (i.e., $F_s = (\sigma_v' K_o \tan\phi) A_{surf}$ where A_{surf} is the surface area for each side of the cap. If a cohesive soil is used for backfill, the undrained strength of the cohesive soil is used in place of $\sigma_v' K_o \tan\phi$. The amount of displacement to mobilize the shear capacity along the side of the cap is usually less than 0.5 in. For many cases, the contributions of side shear are small and can be neglected in the capacity estimate.

Methods used to estimate the load-deformation response of piles are established in Section 10 of the *AASHTO LRFD Bridge Design Specifications* and can be used to develop a stiffness value for the pile group. If liquefaction is possible, appropriate adjustments should be made to evaluate stiffness for the liquefied case. This evaluation involves use of the residual strength of the liquefied soils. Because of uncertainties in the development of liquefaction, checks should also be performed for the nonliquefied case to determine the more critical of the two.



10.6.3.1.2a—Basic Formulation

The nominal bearing resistance shall be estimated using accepted soil mechanics theories and should be based on measured soil parameters. The soil parameters used in the analyses shall be representative of the soil shear strength under the considered loading and subsurface conditions.

The nominal bearing resistance of spread footings on cohesionless soils shall be evaluated using effective stress analyses and drained soil strength parameters.

The nominal bearing resistance of spread footings on cohesive soils shall be evaluated for total stress analyses and undrained soil strength parameters. In cases where the cohesive soils may soften and lose strength with time, the bearing resistance of these soils shall also be evaluated for permanent loading conditions using effective stress analyses and drained soil strength parameters.

For spread footings bearing on compacted soils, the nominal bearing resistance shall be evaluated using the more critical of either total or effective stress analyses.

Except as noted below, the nominal bearing resistance of a soil layer, in ksf, should be taken as:

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\gamma \ B N_{\gamma m} C_{w\gamma} \quad (10.6.3.1.2a\text{-}1)$$

in which:

$$N_{cm} = N_c s_c i_c \quad (10.6.3.1.2a\text{-}2)$$

$$N_{qm} = N_q s_q d_q i_q \quad (10.6.3.1.2a\text{-}3)$$

$$N_\gamma m = N_\gamma s_\gamma i_\gamma \quad (10.6.3.1.2a\text{-}4)$$

where:

c = cohesion, taken as undrained shear strength (ksf)

N_c = cohesion term (undrained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)

N_q = surcharge (embedment) term (drained or undrained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)

10.6.3.1.2b—Considerations for Punching Shear

If local or punching shear failure is possible, the nominal bearing resistance shall be estimated using reduced shear strength parameters c^* and ϕ^* in Eqs. 10.6.3.1.2b-1 and 10.6.3.1.2b-2. The reduced shear parameters may be taken as:

$$c^* = 0.67c \quad (10.6.3.1.2b\text{-}1)$$

$$\phi^* = \tan^{-1}(0.67 \tan \phi_f) \quad (10.6.3.1.2b\text{-}2)$$

where:

c^* = reduced effective stress soil cohesion for punching shear (ksf)

ϕ^* = reduced effective stress soil friction angle for punching shear (degrees)

COWI

PROJECT: **WSDOT I-405 R2B - BR 28W**

CLIENT: **Wood PLC**

CONT: **A207833**

Book: E7

CALC BOOK COMPLETION DATE: **2021 Sep 15**

Title: Pile Caps

Item	Page #'s	to	Page #'s	Subject / Description	Designer's Initials	Checker's Initials	Comments
1	E7-0	-	E7-0	Calc Register (this sheet)	N/A	N/A	
2	E7-1	-	E7-34	Pile Cap Design	RSGR	MEDN	
3	E7-35	-	E7-37	Fin Wall Design	RSGR	MEDN	
4	E7-38	-	E7-40	Curtain Wall Design	RSGR	MEDN	
5	E7-41	-	E7-43	Abutment Seat Length	RSGR	MEDN	
6	E7-44	-	E7-47	Girder Stop	RSGR	MEDN	
7	E7-48		E7-51	Jacking Bearing Capacity and Torsional Check	RSGR	MEDN	
8	E7-52		E7-57	Drawings	RSGR	MEDN	

Pile Cap Design

COWI

PROJECT i405 R2B CONT

SUBJECT B 28 W Pile Cap Design PAGE

CHECKED BY MEDN DATE 2021-SEP-15 CALCULATIONS BY RSGR DATE 2021-SEP-15

Pile Cap design for B28W

Procedure in the design:

1. Pile cap is designed to provide minimum required reinforcement. The minimum reinforcement is then checked against maximum demands from either STR or EXT case (EXT governs)

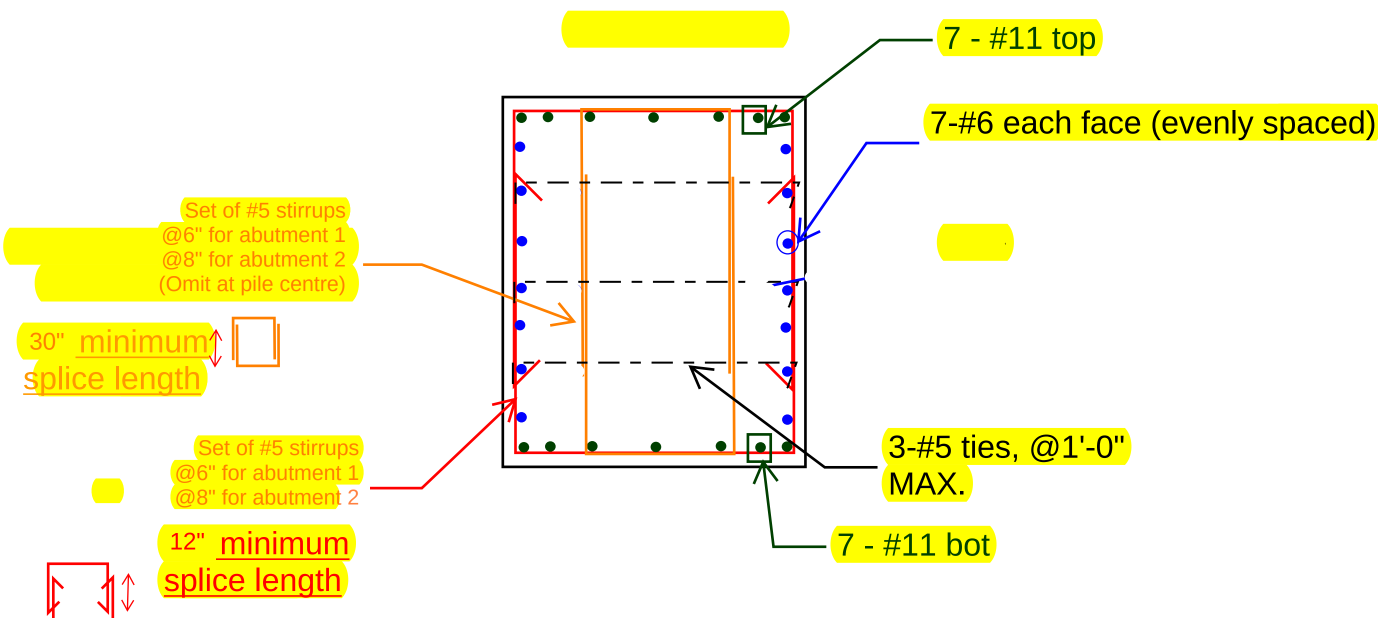
2. Max. flexure and shear demands in pile cap are when the pile top will reach its plastic moment.

3. Shear and torsion demands from Fin Wall have been added to both EXT and STR case.

Total seismic load at abutment 1 = 706 kips
Max. resistance that can be developed by fin wall @1.4" of wall movement is 630 kips (as per geotech addendum) However, piles can resist upto 70 kips x 7 = 490 kips @ 1.5" of movement.
Assuming that piles can only develop 70% of their maximum resistance which occurs @0.8"
At 0.8" of movement fin walls to develop 360 kips of resistance (0.8"/1.4" x 630 kips).

4. Girders more or less sit directly over the piles and hence, do not exert flexural and shear demands into pile cap. The biggest shear and flexure demand from girder is in abutment 1 where the girder sits 2' away from the pile center. See page 12 .

5. Pile cap has been checked against jacking load demands (2 x girder DL) on page 49 to 51.

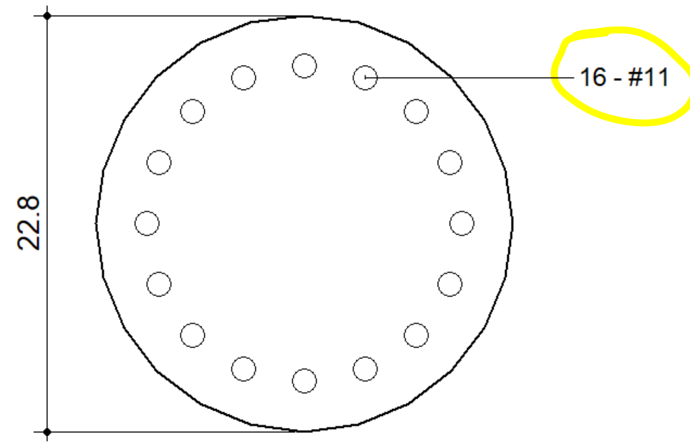


STEP 1: Find out the plastic moment of the pile head.

Using expected material properties

Geometric Properties

	Gross Conc.	Trans (n=6.87)
Area (in^2)	401.8	548.3
Inertia (in^4)	12852.2	18299.1
y_t (in)	11.4	11.4
y_b (in)	11.4	11.4
S_t (in^3)	1129.9	1609.7
S_b (in^3)	1129.9	1607.8



Crack Spacing

2 x dist + 0.1 d_b /ρ

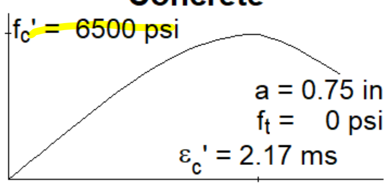
Loading (N,M,V + dN,dM,dV)

-0.0 , 0.0 , 0.0 + 0.0 , 1.0 , 0.0

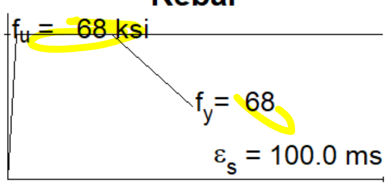
All dimensions in inches

Clear cover to reinforcement = 2.02 in

Concrete



Rebar



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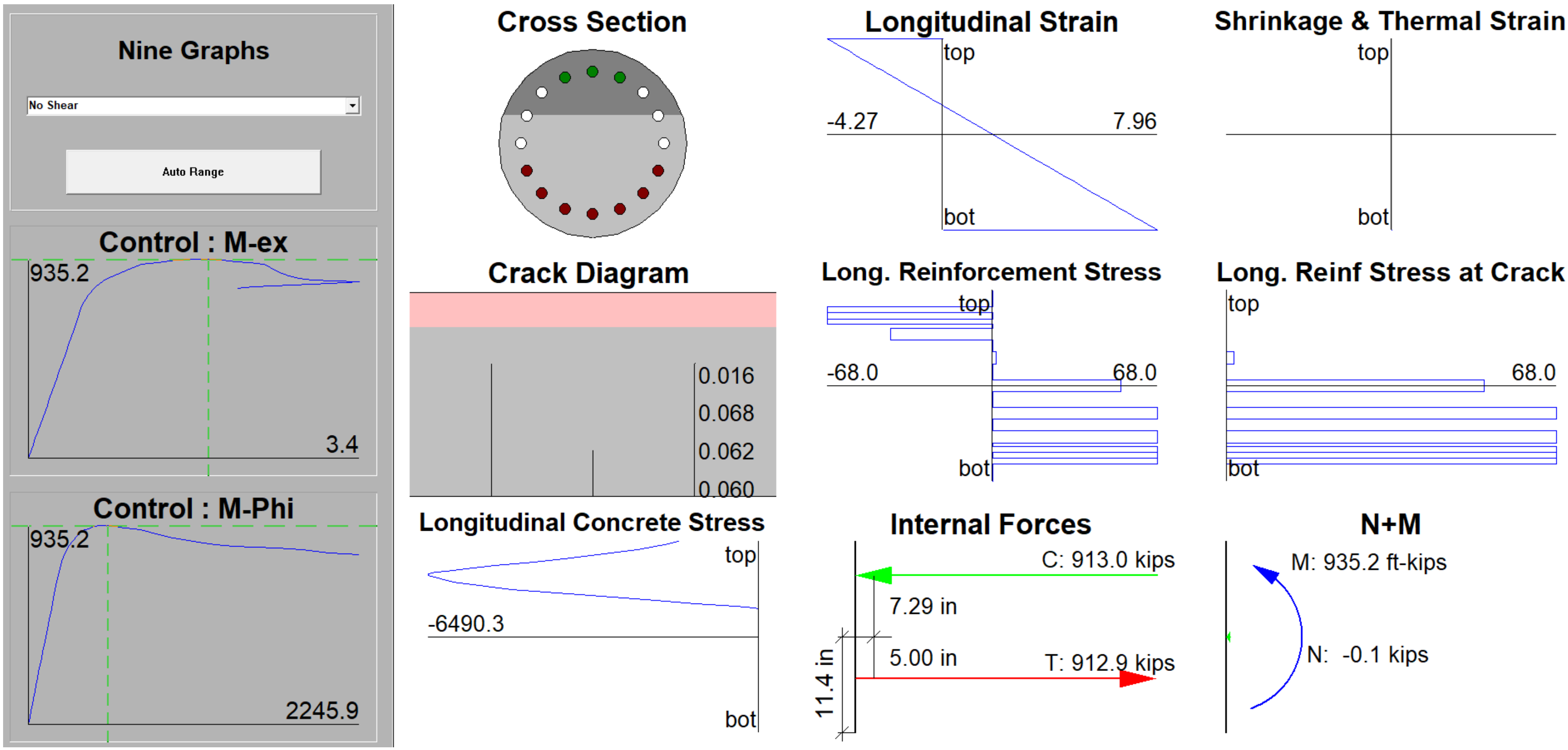
2020/11/13

FOR ABUTMENT 1

STEP 1: Find out the plastic moment of the pile head.

Using expected material properties



FOR ABUTMENT 1

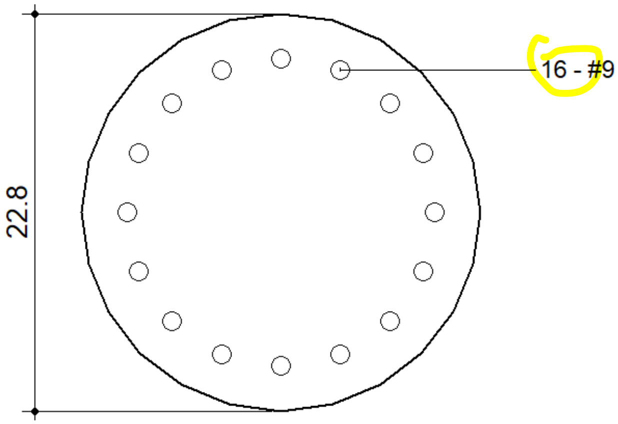
Moving compression force to edge of casing
Mne = 913 x (7.29" + 5" + 4.11")/12 = 1248 k-ft
Overstrength Moment = 1747 k-ft

STEP 1: Find out the plastic moment of the pile head.

Using expected material properties

Geometric Properties

	Gross Conc.	Trans (n=6.87)
Area (in^2)	401.8	495.7
Inertia (in^4)	12852.2	16486.9
y_t (in)	11.4	11.4
y_b (in)	11.4	11.4
S_t (in^3)	1129.9	1450.0
S_b (in^3)	1129.9	1448.8



Crack Spacing

2 x dist + 0.1 d_b /ρ

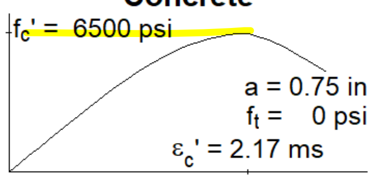
Loading (N,M,V + dN,dM,dV)

-152.0 , 0.0 , 0.0 + 0.0 , 1.0 , 0.0

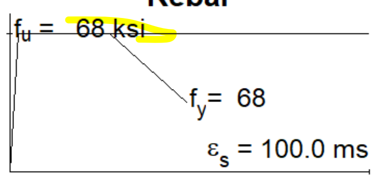
All dimensions in inches

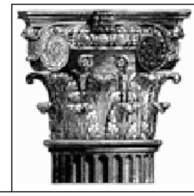
Clear cover to reinforcement = 1.99 in

Concrete



Rebar





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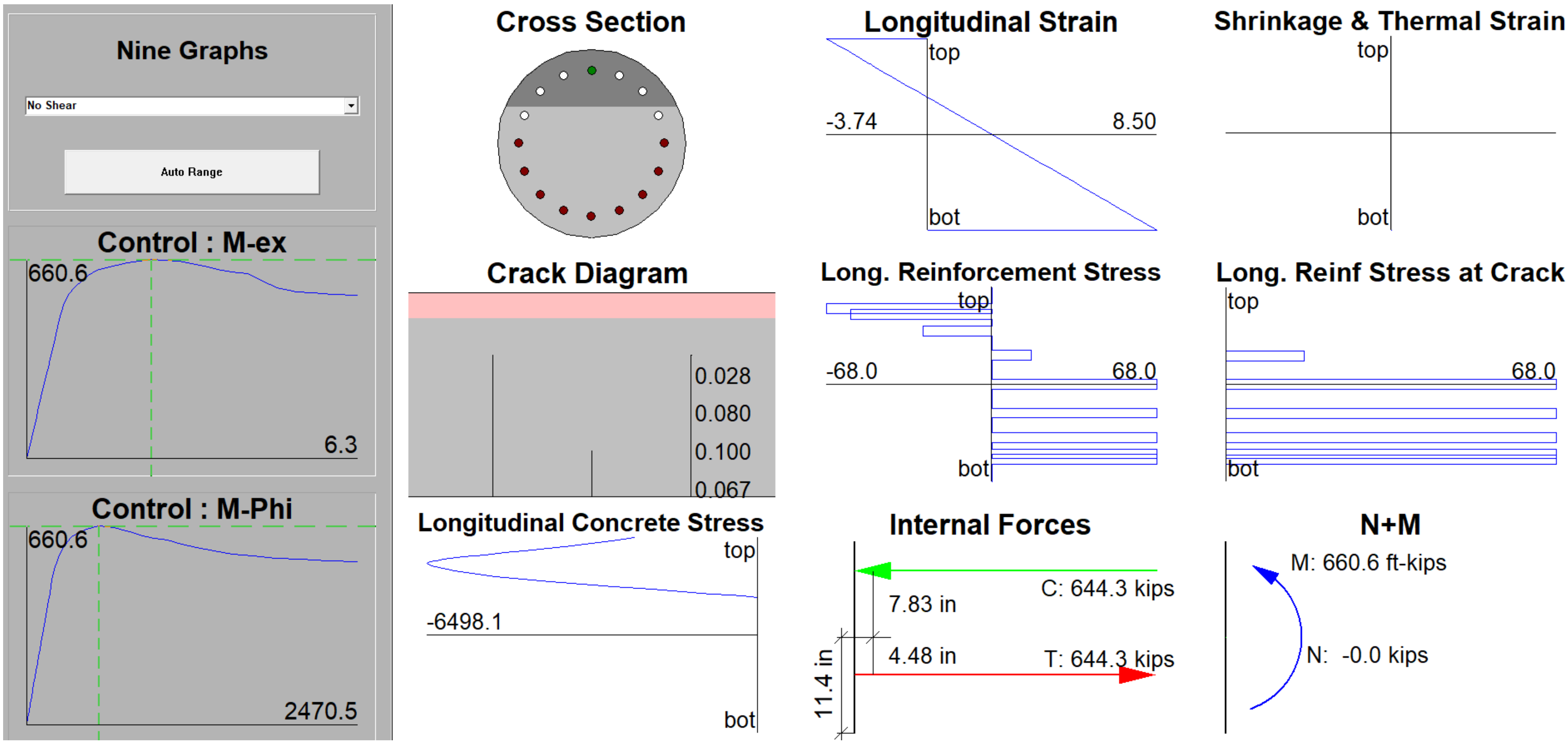
2020/11/13

FOR ABUTMENT 2

STEP 1: Find out the plastic moment of the pile head.

Using expected material properties



FOR ABUTMENT 2

Moving compression force to edge of casing
Mne = 644 x (7.83" + 4.48" + 3.57")/12 = 853 k-ft
Overstrength Moment = 1194 k-ft

STEP 2: DESIGN THE MINIMUM REQ. REINFORCEMENT (TOP AND BOTTOM)

COWI

PROJECT i405 R2B CONT

SUBJECT B28 PILECAP REINFORCEMENT PAGE 01

CHECKED BY MEDN DATE 10 APR 2021 CALCULATIONS BY RSGR DATE 10 APR 2021

$f'_c = 4\,ksi$; $f_y = 60\,ksi$

λ (concrete density factor) = 1.0

$f_r = 0.24 \times 1 \times \sqrt{4} = 0.48\,ksi$

modulus of rupture

$\gamma_3 = 0.67$ (grade 60 steel)

$\gamma_1 = 1.6$

$S_c = \dfrac{48" \times 72" \times 72"}{6} = 41472\ in^3$

D = 72"

b = 48"

MINIMUM REINFORCEMENT IS PROVIDED FOR M_{cr} OF BEAM.

M_{cr} : CRACKING CAPACITY $= \gamma_3 \gamma_1 f_r S_c$

$M_{cr} = 0.67 \times 1.6 \times 0.48 \times 41472 / 12$

$M_{cr} = 1778$ k-ft.

Provide min $\boxed{5\ nos.\ \#11\ bars.}$ $A_s = 7.8\,in^2$

d_e (effective depth) = 72" − 6" − 0.75" − 0.5 × 1.41"

6" ↓ cover to outer layer; 0.75" ↓ outer layer #6; 0.5 × 1.41" ↓ half of #11

$d_e = 64.54"$

$a = \dfrac{7.8 \times 60}{0.85 \times 48 \times 4} = 2.87$

factored Resistance $= \underbrace{0.9}_{\phi_f} \times A_s \times f_y \times \left(\dfrac{d_e - \frac{a}{2}}{12}\right)$

$= 0.9 \times 7.8 \times 60 \times \left(\dfrac{64.54" - 2.87 \times 0.5}{12}\right)$

$\boxed{M_R = 2215\ k\text{-}ft.}$

2215 $M_R > M_{cr}$. 1778

STEP 2: DESIGN THE MINIMUM REQ. TRANS REINFORCEMENT

COWI

PROJECT ______ CONT ______

SUBJECT ______ PAGE 02

CHECKED BY MEDN DATE 2021-SEP-15 CALCULATIONS BY RSGR DATE 2021-SEP-15

MINIMUM TRANSVERSE REINFORCEMENT AASHTO 5.7.2.5 (VERTICAL DIRECTION)

$$A_v \geq 0.0316 \lambda \sqrt{f'_c} \frac{b_v S}{f_y}$$

S: use spacing 8''

$b_v = 48''$ (section width)

$A_v \geq 0.404 \ in^2$

→ $\boxed{\text{1 leg active of \#6}}$

$A_v = 0.44 in^2 \geq 0.404 in^2$

check max. spacing AASHTO 5.7.2.6

v_u = 70 kips / 66" x 48 = 0.02 ksi

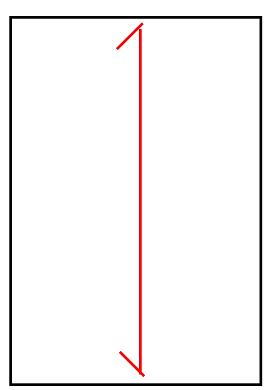
$0.125 f'_c = 0.5$ ksi

Hence $S_{max} = 0.8 d_v \leq 24''$

= 0.8 x (0.72 x 66") = 38"

= Hence, use max. 24''

→ Provided 6" to 8" spacing for stirrups which meets the minimum transverse reinforcement reqiuirement



Min.
Use 1 leg of #6 at 8" spacing
or
2 legs of #5 at 8" spacing

STEP 2: DESIGN THE MINIMUM REQ. SIDE BAR REINFORCEMENT

COWI

PROJECT i 405 CONT

SUBJECT B28 pilecap design PAGE 03

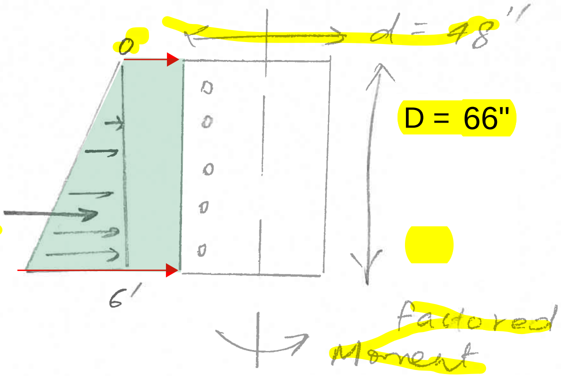
CHECKED BY MEDN DATE 2021-SEP-15 CALCULATIONS BY RSGR DATE 2021-SEP-15

Lateral Bending

Ko = 0.412

at rest

γ_{soil} = 0.135 ksf



Bridge longitudinal direction

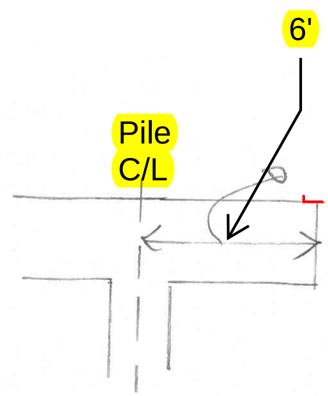
pressure at

At bottom of pile cap = 0.64 ksf (refer to book E6 page 14)

under the diaphragm = 0.42 ksf

Height of pressure = 3.9'

Force on 6' of cantilever (abutment 1) = 2.1 k/ft



distance of cantilever = 6'

moment due to earth pressure

= 2.1 k/ft × 6' x 6' x 0.5

= 38 kips-ft × 1.35 (load factor) = 40 k-ft.

Total lateral moment = 40 k-ft.

side reinforcement = $\boxed{\text{3 nos. \# 6 rebar}}$. A_s = 1.32 in^2

effective depth de = 48" − 3" (cover) − 0.625" (outer bar) − 0.5 × 0.75"

de = 44"

b = 72"

$$a = \frac{A_s \times f_y}{0.85 \times f'_c \times b} = \frac{1.32 \times 60}{0.85 \times 4 \times 66"} = 0.35"$$

STEP 2: DESIGN THE MINIMUM REQ. SIDE BAR REINFORCEMENT

COWI

PROJECT i405 CONT

SUBJECT B28 - pile cap design PAGE 04

CHECKED BY MEDN DATE 2021-SEP-15 CALCULATIONS BY RSGR DATE 2021-SEP-15

Factored resistance = $M_r = \dfrac{\phi_f \; 0.9 \times A_s \times f_y \times \left(d_e - \frac{a}{2}\right)}{12}$

$= \dfrac{0.9 \times 1.32 \times 60 \times \left(44 - \frac{0.35}{2}\right)}{12}$

$M_R = 259$ kip-ft $> M_u$.
40 k-ft.

Hence ok!

Hence, provide minimum side reinforcement is okay! for abutment 2 without Fin Wall

Check side reinforcement in west pile cap (abutment 1) due to maximum moment in Fin Wall on pile cap on page E7. 13

contd. next page.

STEP 2: DESIGN THE MINIMUM HORIZONTAL TIES

COWI

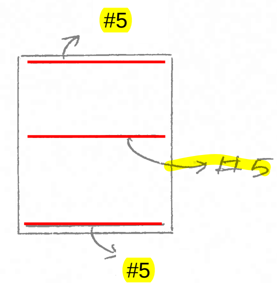
PROJECT ______ CONT ______

SUBJECT ______ PAGE 05

CHECKED BY MEDN DATE 2021-SEP-15 CALCULATIONS BY RSGR DATE 2021-SEP-15

MINIMUM TRANSVERSE REINFORCEMENT (LATERAL DIRECTION)

$$A_v \geq 0.0316\,\lambda \sqrt{f'_c}\,\frac{b_v\,s}{f_y}$$



spacing used for horizontal ties = 8"

b_v = 72"

$A_v \geq 0.607\ in^2$

at pile 2 $\boxed{\begin{array}{l}\text{1 leg of \#5}\\ \text{2 legs of \#5}\end{array}}$ } $A_{v\,prov.}$ = 0.93 in^2

Hence $A_{v\,prov.} > A_v$.

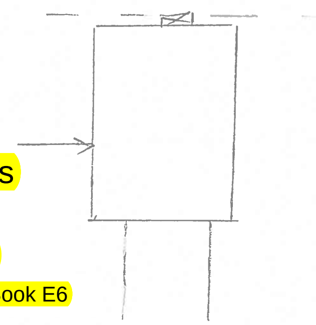
check max spacing AASHTO 5.7.2.6

V_u shear in lateral direction = 70 kips

v_u (stress) = $\dfrac{70\text{ kips}}{48'' \times 66''}$ = 0.02 ksi

70 kips (EXT case)

See page 16 of Book E6



$s_{max} = 0.8\,d_v \leq 24''$

d_v = 0.8 x 0.72 x 48" = 27.65"

use S_{max} = 24".

MINIMUM FLEXURE REINFORCEMENT (SIDES)

Factored M_R = 259 kip-ft.

$1.33\,M_u$ = 1.33 x 40 = 53.2 kip-ft (Mu is the demand from soil pressure in previous page)

$M_{cr} = \gamma_3\,\gamma_1\,f_r\,S_c$

S_c = 27648 in^3

M_{cr} = 1185 kip-ft.

Hence, M_R = 259 $\geq$ min.{53.2, 1185}.

Hence, OK!

COWI

PROJECT ______ CONT ______

SUBJECT ______ PAGE ______

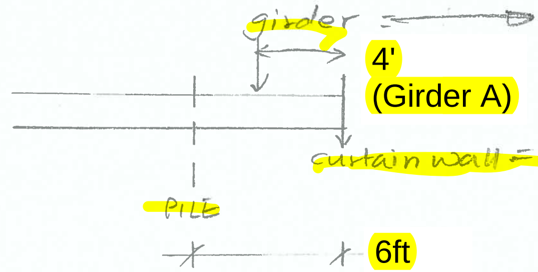
CHECKED BY MEDN DATE 2021-SEP-15 CALCULATIONS BY RSGR DATE 2021-SEP-15

LOCAL CASE

PILE CAP FLEXURE DEMAND.

Case 1 : GIRDER IN OVERHANG. (Occurs in 28W west)

Occurs in Pier 1



girder = DL = 204 kips
0.5 LL = 30 kips

curtain wall = DL = 25 kips

Moment from wall + girder = 234 x (2') + (25 x 6') = 620 k-ft

Moment from pile cap weight = $(0.15 \times 4 \times 6) \times \frac{6'^2}{2}$ = 67 k-ft

Moment from pile going plastic = 1747 k-ft

Total -ve moment at pile location = 620 + 67 + 1747 = 2434 k-ft

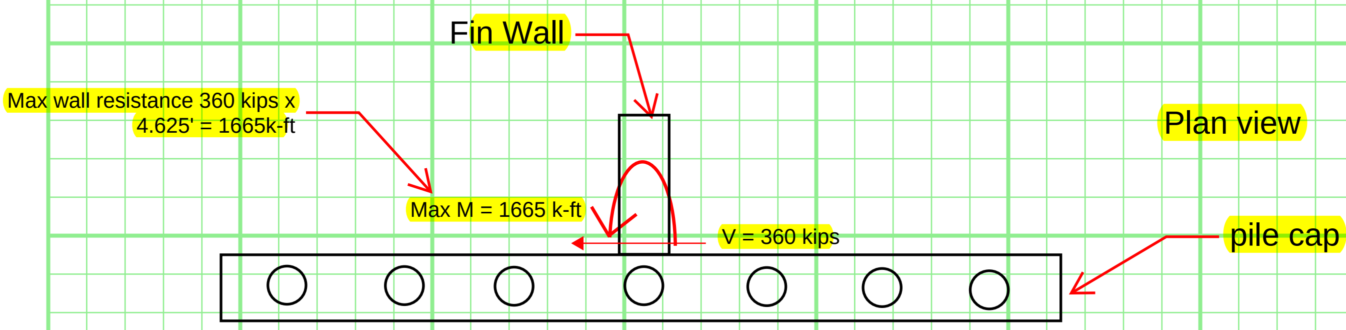
DL & LL, pile cap, Mp

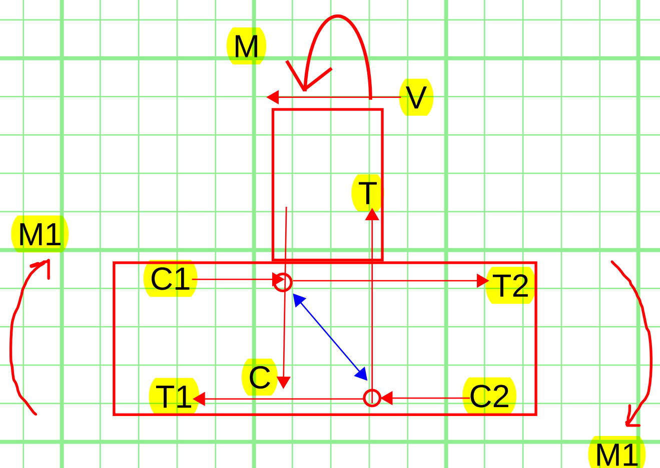
M_u (EXT) = 2434 k-ft

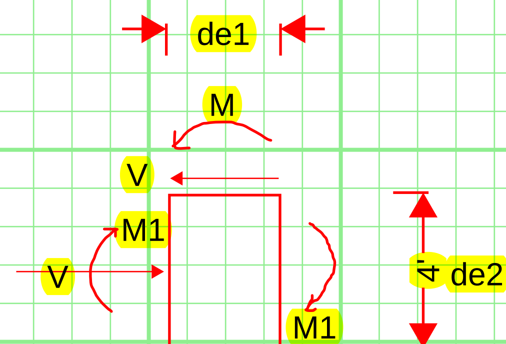
Mr = 3080 k-ft for 7nos. of #11 rebar in Pile cap Top

STEP 2: DESIGN THE MINIMUM REQ. SIDE BAR REINFORCEMENT

Local Check (EXT case) - Load effects of Fin wall to pile cap (for abutment 1 only)







de1 = effective depth in fin wall = 33.295"
de2 = effect depth in pile cap = 44.4"

C = T = M / de1 = 1665 / (33.295"/12) = 600 kips

M + V = 2M1

1665 + 360*2 = 2M1

M1 = 1192 k-ft

Tension at either face of Pile Cap = (M1/de2)= 1192 / (44.4/12) = 322 kips

Area of rebar required in each face = 322 / (0.9 x 68) = 5.26 in2

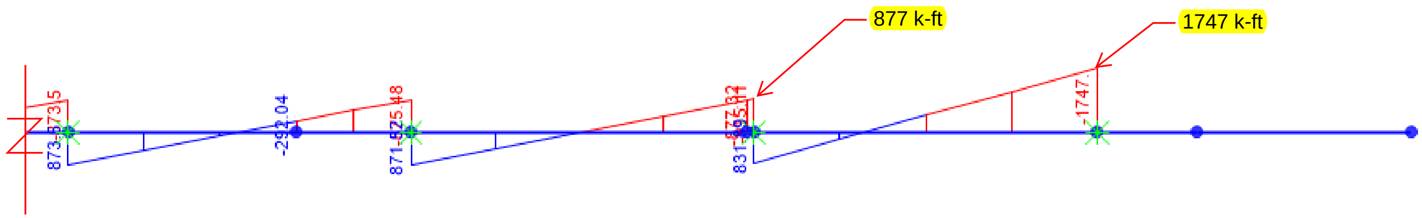
Use #11 rebar = 4.65 / 1.56 = 3 nos. needed
4 nos. #11 provided in Pier 1 below Fin Wall Top Level

STEP 3: CHECK MIN. REINFORCEMENT AGAINST DEMANDS

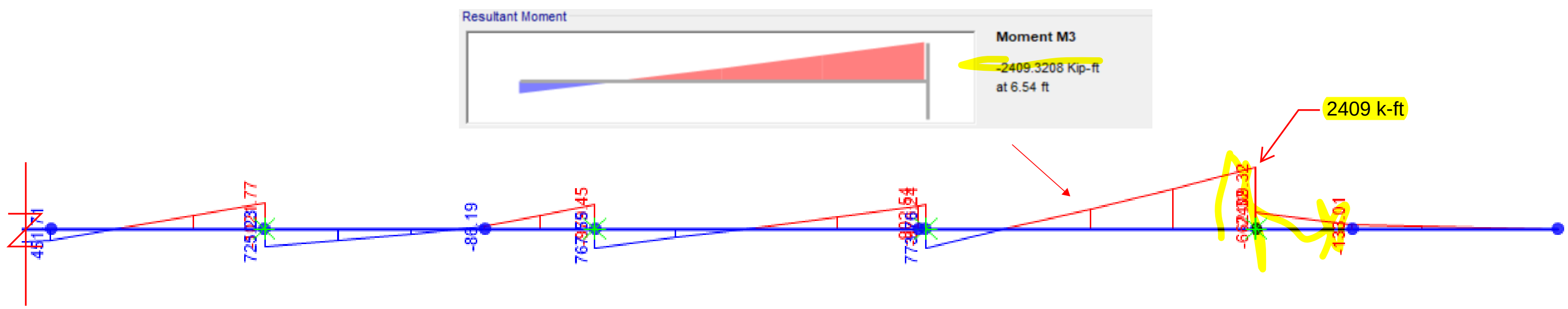
Case 2

GLOBAL CASE

Abutment 1



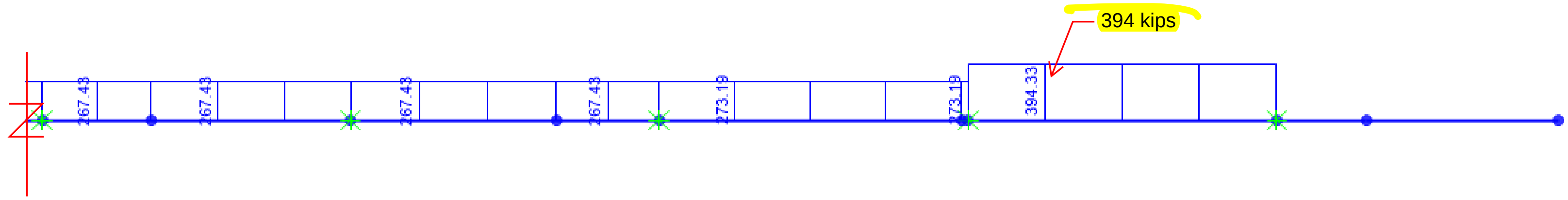
MOMENT IN PILE CAP BEAM DUE TO PLASTIC PILE TOP (REVERSIBLE)



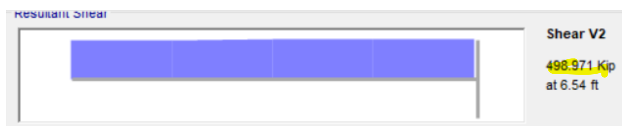
MAX MOMENT IN PILE CAP WHEN COMBINED WITH BEAM SELF WEIGHT AND SUPERSTRUCTURE DL and 0.5 LL

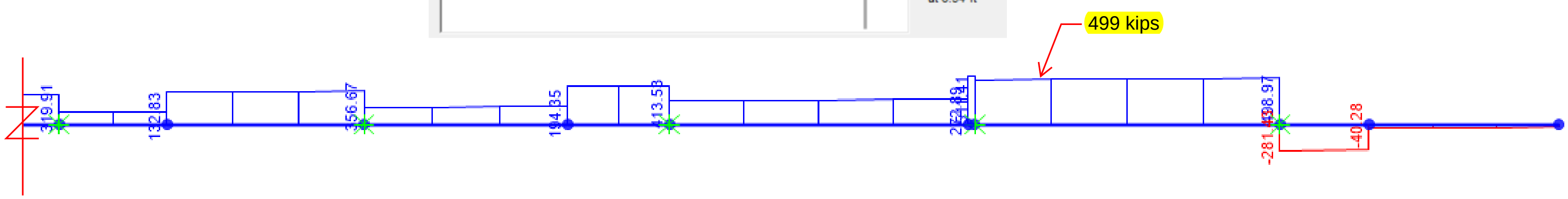
STEP 3: CHECK MIN. REINFORCEMENT AGAINST DEMANDS

Abutment 1



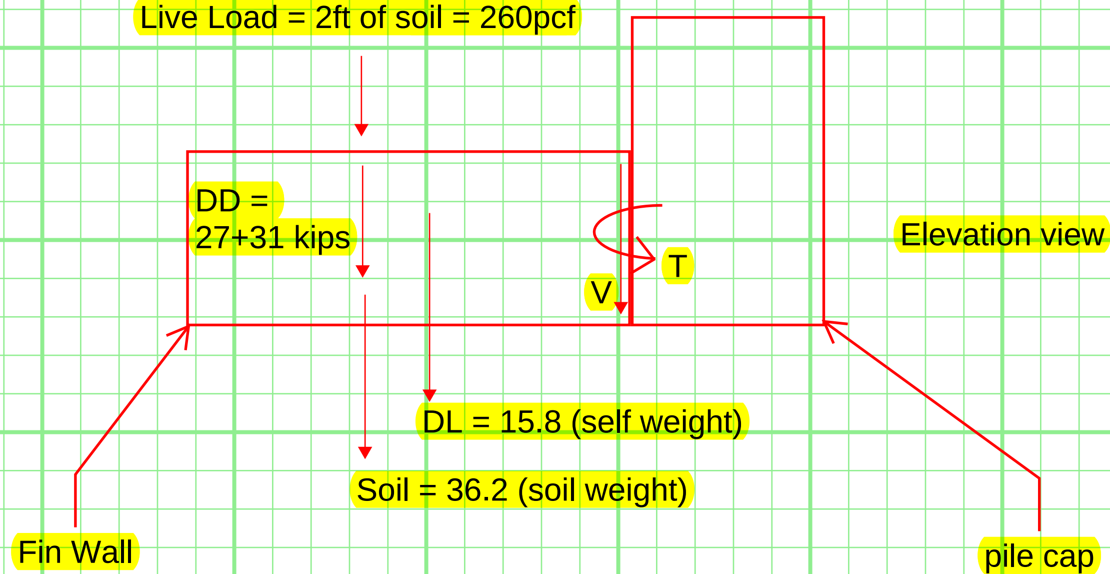
SHEAR IN PILE CAP DUE TO PLASTIC PILE TOP (REVERSIBLE)





MAX SHEAR IN PILE CAP WHEN COMBINED WITH BEAM SELF WEIGHT AND SUPERSTRUCTURE DL and 0.5 LL

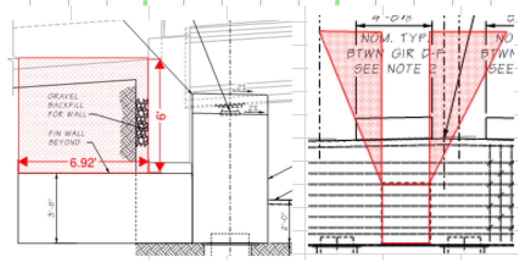
Add to Global Check (EXT case) - Load effects of Fin wall to pile cap



Load effects of Fin wall to pile cap calculated with EXT case factors

Designed Main Top bars:

governing design is STR case



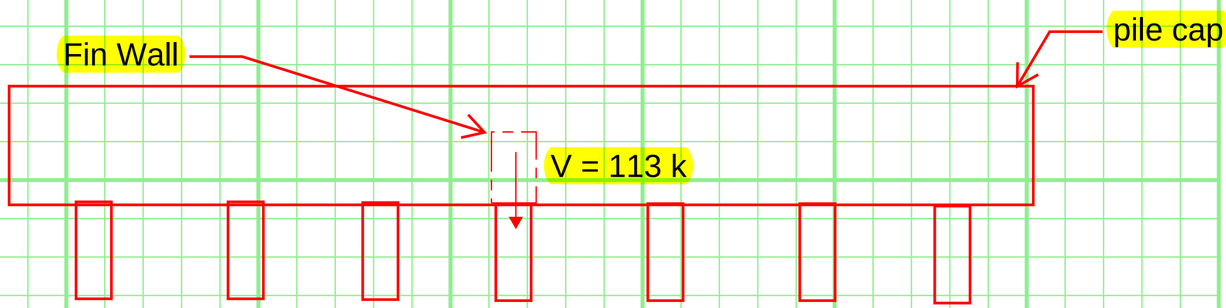
Area of soil in Long Direction	41.5	ft2	=6.92 x 6				
Depth of wall	3	ft					
Volume	124.6	ft3					
Additional Volume	144	ft3	on the sides				
unit weight of soil	0.135	kcf					
Soil Weight	36.2	kips					
			Factor	Factored Load	Moment arm		
Weight of Fin Wall	15.8	kips	1	15.79	4.625	ft	
Soil Load	36.2	kips	1	36.21	5.8	ft	
Net downdrag on Fin wall	58	kips	1	58.00	5.8	ft	
Live Load (assume 2ft of soil height - 260pcf)	5.3976	kips	0.5	2.70	5.8	ft	
			Total, Vertical Load	113	kips		
			Total Moment, M=	635	k-ft		

Fin Wall Load:

Factored Shear, V = 113 kips, Moment at fin-wall location = 635 k-ft (about the bridge transverse direction)

Add this to shear and bending moment effects from superstructure DL and LL

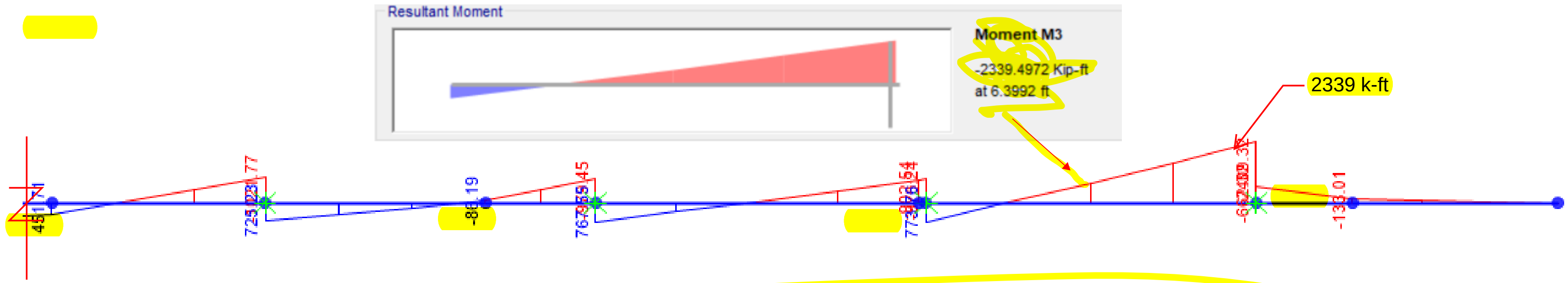
Add to Global Check (EXT case) - Load effects of Fin wall to pile cap



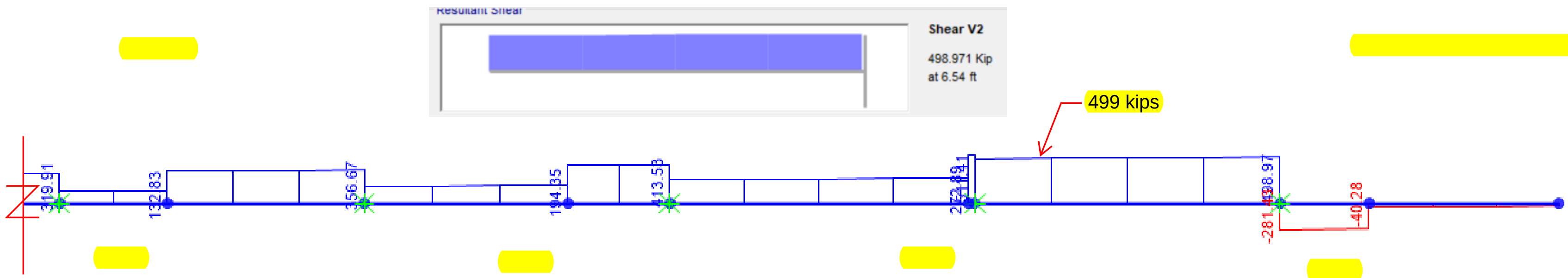
Section view

Torsion in Fin Wall due to differential shear in top and bottom will add to bending moment to pile cap. Add this load effect in addition of superstructure loads and pile Mpo

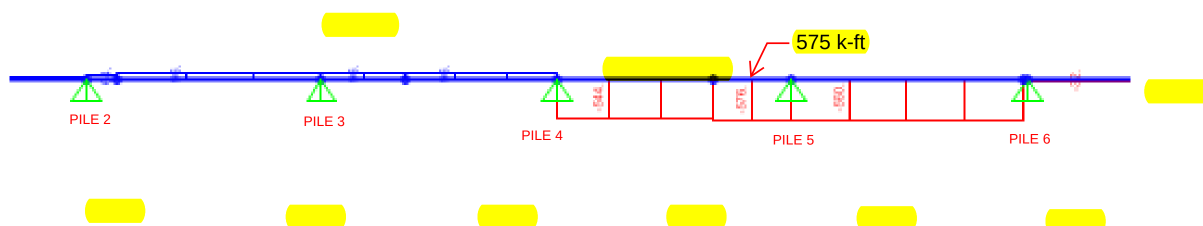
Max Moment in Pile Cap Beam after combining Fin Wall Load Effects (Abutment 1)



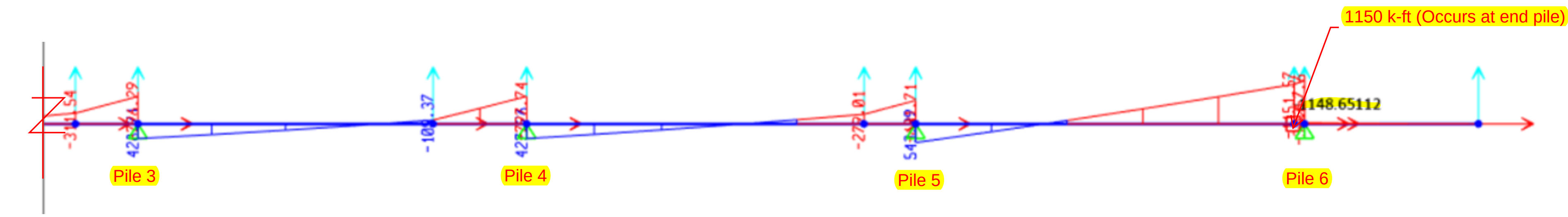
Max Shear in Pile Cap Beam after combining Fin Wall Load Effects (Abutment 1) SAME AS ON PAGE 14



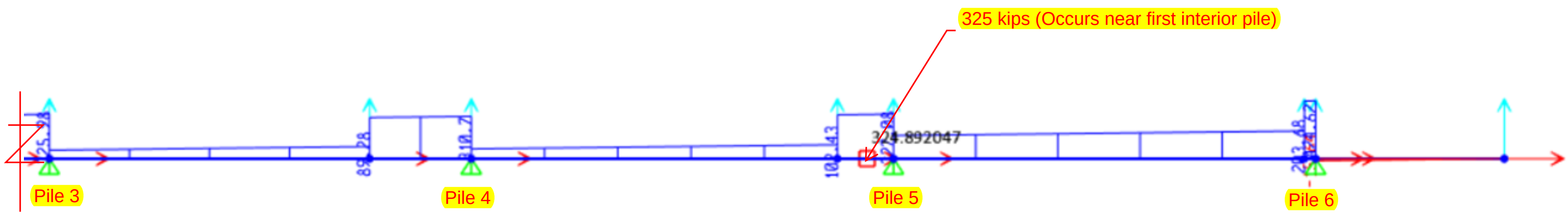
Max Torsion in Pile Cap Beam after combining Fin Wall Load Effects (Abutment 1) SAME AS ON PAGE 14



Max Moment in Pile Cap Beam (Abutment 2) (includes Superstructure, DL and 0.5 LL)



Max Shear in Pile Cap Beam (Abutment 2) (includes Superstructure, DL and 0.5 LL)



STEP 3: CHECK MIN. REINFORCEMENT AGAINST DEMANDS

CASE 3

GLOBAL CASE

Abutment 1 values govern

DESIGN MOMENT : 2339 k-ft

DESIGN SHEAR : 499 kips

DESIGN TORSION : 575 k-ft

Note: Max torsion demand do not occur at same location concurrently with max shear and max moment demands.

COWI

PROJECT:	I-405 R2B Final Design - BR 28 W	JOB NO.:	A207833
SUBJECT:	Pile Cap Design EXT Case		
BY:	RSGR	DATE:	9/13/2021
CHK:	MEDN	DATE:	9/13/2021

SUMMARY

	bar	Nos.	spacing	D/C
Main M- reinforcement at top	11	5	10.34	92%
Main M+ reinforcement at bottom	11	5	10.34	99%
Skin reinforcement each side	6		8.00	-
Transverse reinforcement (2 legs)	5	4	8.00	56%

NOTES AND REFERENCES

λ =	1.00			AASHTO (5.4.2.8-2)
Exposure Level =	Severe			
Strain Gradiant Amplification Factor β =	1.20			
γ =	1.50			
f'_c =	5.20	ksi		(4ksi used for shear and torsion)
w_c =	0.155	kcf		
β_1 =	0.85			
Beam Depth, h =	72.00	in		
Beam Width, b_{wt}=	48.00	in		at top
Beam Width, b_{wb}=	48.00	in		at bottom
Shear Reinforcement?	Yes			
Shear Reinforcement Size #	5		0.625	
Shear Reinforcement Spacing	8.00	in		
# of Shear Reinforcement Legs	4.00			
Clear Cover (c_c) =	6.00	in		
f_y =	68.00	ksi		(60 ksi used for shear and torsion)
ε_{ty} ($\varepsilon_{balanced}$) =	0.00	in/in		AASHTO 5.6.2.1
$\varepsilon_{tension\ controlled}$ =	0.01	in/in		AASHTO 5.6.2.1
ρ_{min} =	0.00			
ρ_{max} =	0.03			

Designed Main bars:

bar #		11.00	
bar diam	db	1.41	in
bar area	Ab	1.56	in2

Demands (Taken at face of pile)

	STR Case		M (k-ft)	T (k-ft)	V (kips)
Max +Moment Case			2339	0	499
Max -Moment Case			-2339	0	499
Max Torsion Case			-979	575	413
Max Shear Case			-2339	50	499

from SAP

The effective is calculated as follows:

Cover top	2	in	
Cover bottom	6	in	ignore bottom 6"
de top reinforcement	68.67	in	
de bottom reinforcement	64.67	in	

Solve for the required amount of reinforcing steel, as follows:

	φf=		0.9						AASHTO 5.5.4.2
	b=		48.00	in					width
	fc=		5.2	ksi					
	Max -M =		2339	k-ft	Max +M	2339	k-ft		
	Rn=		0.1378	ksi		0.1554	ksi		
	ρ=		0.00206			0.00233			
	As,req=		6.78619	in2		7.22103	in2		
	Nos. prov	nos.	5			5			
	Bar size	#	11			11			
	Bar Area		1.56	in2		1.56	in2		
	$A_{s,prov}$ =		7.80	in2		7.80	in2		
	Spacing, s=		10.34	in		10.34	in		
	Use		10	in		10	in		Spacing provided
	a =		10.00	in		10.00	in		Depth of rectangular compression block
	$\varphi_f M_n$ =		2532.79	k-ft	OK	2373.67	k-ft	OK	Flexural capacity
strain compatibility check	c=		1.47	in	OK	1.28	in	OK	AASHTO 5.7.3.1.1-4; 5.7.2.1-1
tensile stress check	et=		0.14		OK	0.16		OK	

$$Rn = \frac{Mu_{cap\_str1} \cdot 12\frac{in}{ft}}{\left(\phi_f \cdot b \cdot d_e^{\,2}\right)}$$

$$\rho = 0.85\left(\frac{f_c}{f_y}\right)\left[1.0 - \sqrt{1.0 - \frac{(2 \cdot Rn)}{(0.85 \cdot f_c)}}\right]$$

COWI

PROJECT:	I-405 R2B Final Design - BR 28 W	JOB NO.:	A207833
SUBJECT:	Pile Cap Design EXT Case		
BY:	RSGR	DATE:	9/13/2021
CHK:	MEDN	DATE:	9/13/2021

The minimum reinforcement requirements will be calculated for the cap.

The cracking strength is calculated as follows:

fr=	0.48	ksi			AASHTO 5.4.2.6
Ig=	1492992.00	in4			
yt=	36.00	in			
γ1=	1.60				AASHTO 5.6.3.3
γ3=	0.67				AASHTO 5.6.3.3
Sc=	41472.00	in3			
Mcr=	1778.32	k-ft			AASHTO eq 5.6.3.3-1
$\varphi_f M_{n(-)}$ =	2533		>	1778	OK
$\varphi_f M_{n(+)}$ =	2374		>	1778	OK

$f_r = 0.24 \cdot \sqrt{f_c}$

$$M_{cr} = \gamma_3 \left[\left(\gamma_1 f_r + \gamma_2 f_{cpe} \right) S_c - M_{dnc} \left(\frac{S_c}{S_{nc}} - 1 \right) \right] \quad (5.6.3.3\text{-}1)$$

Design for Flexure (Service I)

SEE PAGE 27 and 28

Design for Flexure (Skin Reinforcement): longitudinal skin reinforcement

additional longitudinal steel must be provided along the side faces of concrete members deeper than three feet.

this check is carried out using the effective depth (d_e) and the required longitudinal tension steel in place of specific applied factored loads

					distance from ext comp fiber to centroid of ext tension steel (.in)
de=	64.67	in			
As_cap=	7.80	in2			
Ask=	0.42	in2		Lower limit each side	AASHTO eq 5.6.7-3
Ask=	3.90	in2		Upper limit each side	
bar used=	6.00				
bar area=	0.75	in2			
spacing used=	8.00	in	OK	10.77833333 =Max Spacing	AASHTO 5.6.7
Ask Prov=	4.85	in2	OK		Less than upper limit each side

Design for Shear and Torsion (Strength I)

The presence of torsion affects the total required amount of both longitudinal and transverse reinforcing steel. However, if the applied torsion is less than one-quarter of the factored torsional cracking moment, then the Specifications allow the applied torsion to be ignored. This computation is shown as follows:

φt=	0.90			Resistance factor for shear and torsion
Area of concrete Acp=	3456.00	in2		
Perimeter Pc=	240.00	in		
K=	2.00			
Tcr=	2073.60	k-ft		AASHTO eq 5.7.2.1-4
0.25 x φt x Tcr=	466.56	k-ft	Need to Design for Torsion	Torsion Threshold AASHTO eq 5.7.2.1-3

First Design the shear reinforcement and then check for torsional resistance

nominal shear resistance of the critical section is a combination of the nominal resistance of the concrete and the nominal resistance of the steel. This value is then compared to a computed upper-bound value and the lesser of the two controls. These calculations are illustrated below:

Shear bar size=	5			
Number of legs=	4			
Spacing of Shear bars, S=	8	in		
Clear Spacing=	7.375	in		
Av =	1.24	in^2	=4 legs of #5	
dv=	71.95	in	effective shear depth	AASHTO C5.7.2.8-1
Use dv=	71.95	in		
β=	2.00			AASHTO 5.7.3.4.1
Theta=	45.00	deg		AASHTO 5.7.3.4.1
Use β =	1.22			AASHTO eq 5.7.3.4.2-1
Use ϴ =	42.76			AASHTO eq 5.7.3.4.2-3
Vc=	265.36	kips		AASHTO eq 5.7.3.3-3
Shear Threshold=	119.41	kips		AASHTO eq 5.7.2.3-1
Transverse Reinforcement Required				
A_v min =	0.40	in2		AASHTO eq 5.7.2.5-1
v_u =	0.16	ksi		Shear Stress AASHTO eq 5.7.2.8-1
s max =	24.00	in		AASHTO 5.7.2.6
e_s =	0.00			AASHTO eq 5.7.3.4.2-4
V_s =	725.78	kips		AASHTO eq 5.7.3.3-4
$\Phi V_n = V_r$ =	892.03	OK	D/C 56%	

COWI	PROJECT: I-405 R2B Final Design - BR 28 W	JOB NO.: A207833
	SUBJECT: Pile Cap Design EXT Case	
	BY: RSGR	DATE: 9/13/2021
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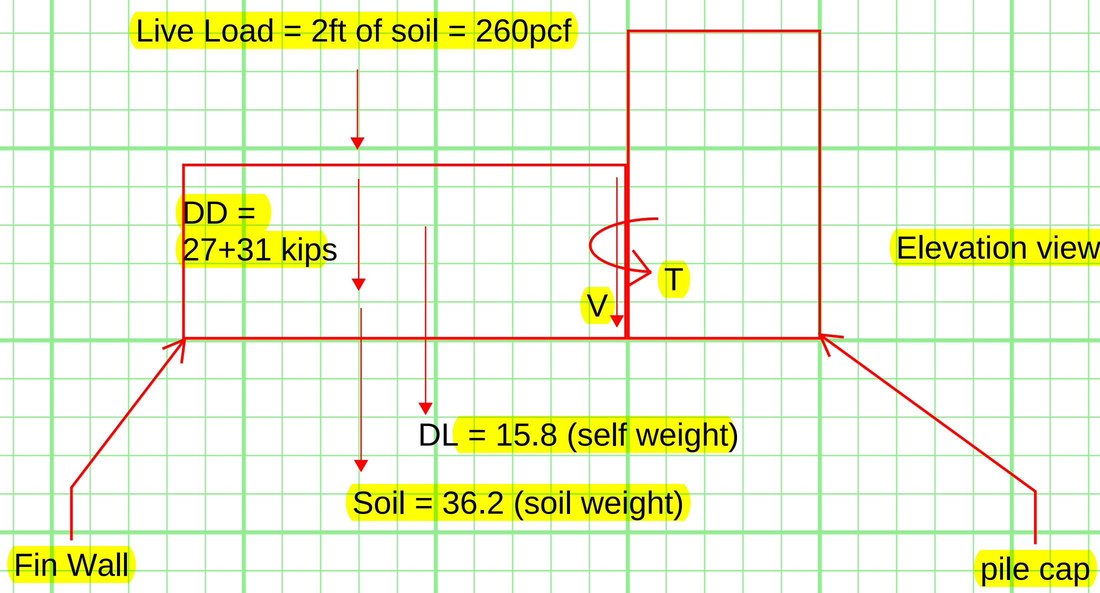
Check for torsional resistance

A_{cp} =	3456.00	in^2		
p_c =	240.00	in		
A_o =	2189.08	in^2	Area enclosed by shear path	85% of the hoop
p_h =	205.50	in		
Equivalent V_u =	505.50	kips		AASHTO eq 5.7.3.4.2-5
v_u =	0.16	ksi	shear stress	
s max =	24.00	in		
e_s =	0.00			AASHTO eq 5.7.3.4.2-4
β =	1.72			AASHTO eq 5.7.3.4.2-1
Θ =	37.32	degrees		AASHTO eq 5.7.3.4.2-3
V_c =	376.33	kips		AASHTO eq 5.7.3.3-3
V_s =	880.13	kips		AASHTO eq 5.7.3.3-4
$\Phi V_n = V_r$ =	1130.82	kips		
A_t =	0.31	in^2		Area of transverse rebar
T_n =	1115.80	k-ft		AASHTO eq 5.7.3.6.2-1
$\Phi T_n = T_r$	1004.22	k-ft		
A_t / s - Req.	0.026	in^2 / in		
A_t / s - Prov.	0.039	in^2 / in	OK	D/C= 67%
Longitudinal A_s - Req.	9.14	in^2		AASHTO eq 5.7.3.6.3-1
A_s - Provided	15.60	in^2	OK	(7.8in^2 for 5nos. of #11 rebar) x 2 (top and bottom layer)

Global Check (STR case) - Load effects of Fin wall added to pile cap

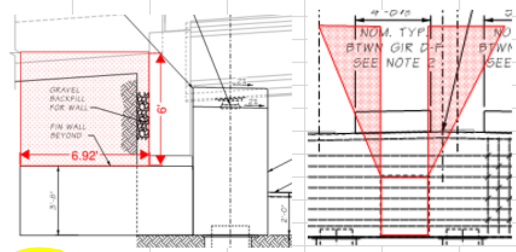


Load effects of Fin wall to pile cap calculated in Fin wall reinforcement design sheet

Designed Main Top bars:

governing design is STR case

Area of soil in Long Direction	41.5	ft2	=6.92 x 6			
Depth of wall	3	ft				
Volume	124.6	ft3				
Additional Volume	144	ft3	on the sides			
unit weight of soil	0.135	kcf				
Soil Weight	36.2	kips				
			Factor	Factored Load	Moment arm	
Weight of Fin Wall	15.8	kips	1.25	19.73	4.625	ft
Soil Load	36.2	kips	1.3	47.07	5.8	ft
Net downdrag on Fin wall	58	kips	1.4	81.20	5.8	ft
Live Load (assume 2ft of soil height - 260pcf)	5.3976	kips	1.75	9.45	5.8	ft
			Total, Vertical Load	157	kips	
			Total Moment, M=	**890**	k-ft	



From FIN Wall reinforcement calc sheets

Fin Wall Load:

Factored Shear, V = 157 kips

Factored Moment = Torsion to pile cap, T = 890 kips (anti-clockwise)

Add these to shear and bending moment effects from superstructure DL and LL into STR Stick model (next page)

COWI	PROJECT: I-405 R2B Final Design - BR 28	JOB NO.: A207833
	SUBJECT: Pile Cap Design Check to add Fin Wall shear and Torsion to STR case	
	BY: RSGR	DATE: 9/13/2021
	CHK: MEDN	DATE: 9/13/2021

SUMMARY

	bar	Nos.	spacing	D/C
Main M- reinforcement at top	11	5	10.34	43%
Main M+ reinforcement at bottom	11	5	10.34	16%
Transverse reinforcement (2 legs)	5	4	8.00	59%

NOTES AND REFERENCES

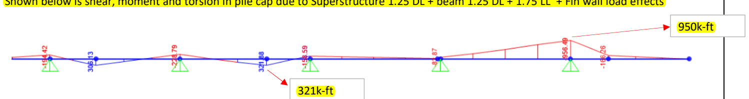
			Notes
λ =	1.00		AASHTO (5.4.2.8-2)
Exposure Level =	Severe		
Strain Gradiant Amplification Factor β =	1.20		
γ =	1.50		
f'_c =	4.00	ksi	
w_c =	0.155	kcf	
β_1 =	0.85		
Beam Depth, h =	72.00	in	
Beam Width, b_{wt}=	48.00	in	at top
Beam Width, b_{wb}=	48.00	in	at bottom
Shear Reinforcement?	Yes		
Shear Reinforcement Size #	5	0.625	
Shear Reinforcement Spacing	8.00	in	
# of Shear Reinforcement Legs	4.00		
Clear Cover (c_c) =	6.00	in	
f_y =	60.00	ksi	
ε_{ty} ($\varepsilon_{balanced}$) =	0.00	in/in	AASHTO 5.6.2.1
$\varepsilon_{tension\ controlled}$ =	0.01	in/in	AASHTO 5.6.2.1
ρ_{min} =	0.00		
ρ_{max} =	0.02		

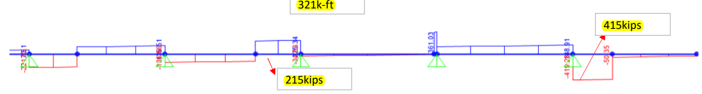
Designed Main bars:

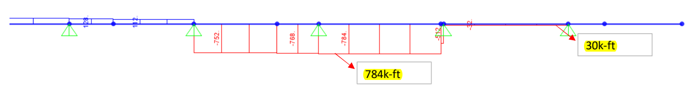
bar	11.00	
bar diam	1.41	in
bar area	1.56	in2

Demands (Taken at face of pile)

Shown below is shear, moment and torsion in pile cap due to Superstructure 1.25 DL + beam 1.25 DL + 1.75 LL + Fin wall load effects







	STR Case		M (k-ft)	T (k-ft)	V (kips)
Max +Moment Case			321	784	215
Max -Moment Case			-950	30	415
Max Torsion Case			321	784	215
Max Shear Case			-950	30	415

from SAP + Fin wall case

The effective is calculated as follows:

Cover top	2	in
Cover bottom	6	in
de top reinforcement	68.67	in
de bottom reinforcement	64.67	in

Solve for the required amount of reinforcing steel, as follows:

φf=	0.9		STR case				AASHTO 5.5.4.2	
b=	48.00	in					top	
fc=	4	ksi						
Max -M =	950	k-ft		Max +M	321	k-ft		
Rn=	0.0560	ksi			0.0213	ksi		
ρ=	0.00094				0.00036			
As,req=	3.10001	in2			1.10652	in2		
Nos. prov	5.00000				5.00000			
Bar size	11.00000				11.00000			
Bar Area	1.56000	in2			1.56000	in2		
$A_{s,prov}$ =	7.80000	in2			7.80000	in2		
Spacing, s=	10.33500	in			10.33500	in		
Use	10.00000	in			10.00000	in		Spacing provided
a =	11.47059	in			11.47059	in		Depth of rectangular compression block
$\varphi_f M_n$ =	2209.0	k-ft	OK		2069	k-ft	OK	Flexural capacity
strain compatibility check c=	2.15917	in	OK		1.88927	in	OK	AASHTO 5.7.3.1.1-4; 5.7.2.1-1
tensile stress check et=	0.09241		OK		0.10604		OK	

$$Rn = \frac{Mu_{cap\_str1} \cdot 12 \frac{in}{ft}}{\left(\phi_f \cdot b \cdot d_e^2\right)}$$

$$\rho = 0.85\left(\frac{f_c}{f_y}\right)\left[1.0 - \sqrt{1.0 - \frac{(2 \cdot Rn)}{(0.85 \cdot f_c)}}\right]$$

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	SUBJECT:	Pile Cap Design Check to add Fin Wall shear and Torsion to STR case		
	BY:	RSGR	DATE:	9/13/2021
	CHK:	MEDN	DATE:	9/13/2021

The minimum reinforcement requirements will be calculated for the cap.

The cracking strength is calculated as follows:

fr=	0.48	ksi			AASHTO 5.4.2.6
Ig=	1492992.00	in4			
yt=	36.00	in			
γ1=	1.60				AASHTO 5.6.3.3
γ3=	0.67				AASHTO 5.6.3.3
Sc=	41472.00	in3			
Mcr=	1778.32	k-ft			AASHTO eq 5.6.3.3-1
$\varphi_f M_{n(-)}$ =	2209		>	1264	OK
$\varphi_f M_{n(+)}$ =	2069		>	427	OK

$$f_r = 0.24 \cdot \sqrt{f_c}$$

$$M_{cr} = \gamma_3 \left[\left(\gamma_1 f_r + \gamma_2 f_{cpe} \right) S_c - M_{dnc} \left(\frac{S_c}{S_{nc}} - 1 \right) \right] \quad (5.6.3.3\text{-}1)$$

Design for Shear and Torsion

The presence of torsion affects the total required amount of both longitudinal and transverse reinforcing steel. However, if the applied torsion is less than one-quarter of the factored torsional cracking moment, then the Specifications allow the applied torsion to be ignored. This computation is shown as follows:

φt=	0.90			Resistance factor for shear and torsion
Area of concrete Acp=	3456.00	in2		
Perimeter Pc=	240.00	in		
K=	2.00			
Tcr=	2073.60	k-ft		AASHTO eq 5.7.2.1-4
0.25 x φt x Tcr=	466.56	k-ft	Need to Check for Torsion	Torsion Threshold AASHTO eq 5.7.2.1-3

First Design the shear reinforcement and then check for torsional resistance

nominal shear resistance of the critical section is a combination of the nominal resistance of the concrete and the nominal resistance of the steel. This value is then compared to a computed upper-bound value and the lesser of the two controls. These calculations are illustrated below:

Shear bar size=	5			
Number of legs=	4			
Spacing of Shear bars, S=	8	in		
Clear Spacing=	7.375	in		
Av =	1.24	in^2		
dv=	62.93	in	effective shear depth	AASHTO C5.7.2.8-1
Use dv=	62.93	in		
β=	2.00			AASHTO 5.7.3.4.1
Theta=	45.00	deg		AASHTO 5.7.3.4.1
Use β =	1.61			AASHTO eq 5.7.3.4.2-1
Use ϴ =	38.22			AASHTO eq 5.7.3.4.2-3
Vc=	307.87	kips		AASHTO eq 5.7.3.3-3
Shear Threshold=	138.54	kips		AASHTO eq 5.7.2.3-1
	Transverse Reinforcement Required			
A_v min =	0.40	in2		AASHTO eq 5.7.2.5-1
v_u =	0.15	ksi		Shear Stress AASHTO eq 5.7.2.8-1
s max =	24.00	in		AASHTO 5.7.2.6
e_s =	0.00			AASHTO eq 5.7.3.4.2-4
V_s =	743.13	kips		AASHTO eq 5.7.3.3-4
$\Phi V_n = V_r$ =	945.90	OK		

Check for torsional resistance

A_{cp} =	3456.00	in^2		
p_c =	240.00	in		
A_o =	2189.08	in^2	Area enclosed by shear path	85% of the hoop
p_h =	205.50	in		
Equivalent V_u =	451.86	kips		AASHTO eq 5.7.3.4.2-5
v_u =	0.17	ksi	shear stress	
s max =	24.00	in		
e_s =	0.00			AASHTO eq 5.7.3.4.2-4
β =	1.91			AASHTO eq 5.7.3.4.2-1
ϴ =	36.07	degrees		AASHTO eq 5.7.3.4.2-3
V_c =	364.36	kips		AASHTO eq 5.7.3.3-3
V_s =	803.50	kips		AASHTO eq 5.7.3.3-4
$\Phi V_n = V_r$ =	1051.08	kips		
A_t =	0.31	in^2		Area of transverse rebar
T_n =	1164.53	k-ft		AASHTO eq 5.7.3.6.2-1
$\Phi T_n = T_r$	1048.07	k-ft		
A_t / s - Req.	0.023	in^2 / in		
A_t / s - Prov.	0.039	in^2 / in	OK	D/C= 59%
Longitudinal A_s - Req.	9.10	in^2		AASHTO eq 5.7.3.6.3-1
A_s - Provided	15.60	in^2	OK	($7.8in^2$ for 5nos. of #11 rebar) x 2 (top and bottom layer)

COWI	PROJECT: I-405 R2B Final Design - BR 28 W	JOB NO.:	A207833
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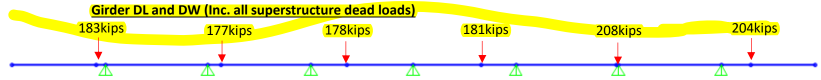
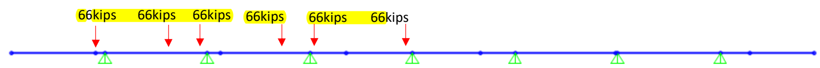
NOTES AND REFERENCES

Girder DL and DW (Inc. all superstructure dead loads)

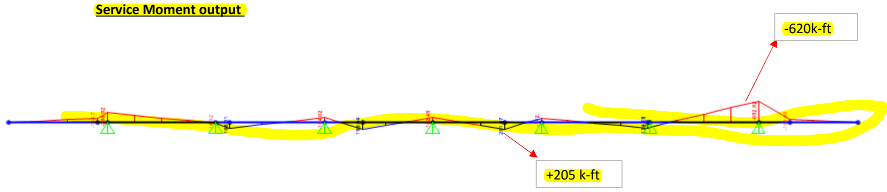


LL

Controlling case - 3 Lane case (for HL-93) - Includes DLA and multiply by MPF to the value below (Moved transversly)



Service Moment output



λ =		1.00		AASHTO (5.4.2.8-2)
Exposure Level =		Severe		
Strain Gradiant Amplification Factor β =		1.20		
γ =		1.50		
f'_c =		4.00	ksi	
w_c =		0.155	kcf	
β_1 =		0.85		
Beam Depth, h =		72.00	in	used 72" for service case check
Beam Width, b_{wt}=		48.00	in	at top
Beam Width, b_{wb}=		48.00	in	at bottom
Shear Reinforcement?		Yes		
Shear Reinforcement Size	#	5	0.625	
Shear Reinforcement Spacing		8.00	in	
# of Shear Reinforcement Legs		2.00		
Clear Cover (c_c) =		6.00	in	
f_y =		60.00	ksi	
ε_{ty} ($\varepsilon_{balanced}$) =		0.002	in/in	AASHTO 5.6.2.1
$\varepsilon_{tension\ controlled}$ =		0.005	in/in	AASHTO 5.6.2.1
ρ_{min} =		0.003		
ρ_{max} =		0.021		

Designed Main bars:

bar	11.00	
bar diam	1.41	in
bar area	1.56	in2

Demands (Taken at face of pile)

	SER Case		M (k-ft)	T (k-ft)	V (kips)	
Max Moment Case			-620	0	0	from SAP
Max +Moment Case	Case3_SER1		205	0	0	

The effective is calculated as follows:

Cover top	2	in
Cover bottom	6	in
de top reinforcement	68.67	in
de bottom reinforcement	64.67	in

COWI	**PROJECT:** I-405 R2B Final Design - BR 28 W		**JOB NO.:** A207833
	SUBJECT: Pile Cap Service Design		
	BY: RSGR		**DATE:** 9/13/2021
	CHK: MEDN		**DATE:** 9/13/2021

Design for Flexure (Service I)

The control of cracking by distribution of reinforcement must be satisfied.

Negative Reinforcement			Positive Reinforcement			
h =	72.00		h =	72.00		Beam Depth
d_c =	3.33		d_c =	7.33		Distance from extreme tension fiber to to center of flexural reinforcement
β_s =	1.07		β_s =	1.16		AASHTO Eq. 5.6.7-2
Υ_e =	0.75		Υ_e =	0.75		Exposure Class 2 - Increased concern of corrosion
s =	7.00		s =	7.00		Spacing of reinforcement
$f_{ss,\ allow}$ =	35.94	ksi	$f_{ss,\ allow}$ =	20.86	ksi	Allowable tensile stress in reinforcement, AASHTO Eq. 5.6.7-1
Mu_Ser	620.00	k-ft	Mu_Ser	205.00	k-ft	
ρ =	0.002		ρ =	0.002		Reinforcement Ratio (As, prov / b x d) used 5 nos. of #11 rebars
Es =	29000.00	ksi	Es =	29000.00	ksi	
Ec =	4027.56		Ec =	4027.56		AASHTO (C5.4.2.4-2)
n =	7.20		n =	7.20		Modular Ration
k =	0.16		k =	0.16		$k = \sqrt{(\rho \cdot n)^2 + (2 \cdot \rho \cdot n)} - \rho \cdot n$
c = kd_e =	11.31		c = kd_e =	10.66		Depth to N.A.
y =	57.36		y =	54.01		Lever Arm
I_t =	207932.56	in4	I_t =	183216.92	in4	Transformed moment of inertia
f_s =	14.78	OK	f_s =	5.22	OK	Stress in the reinforcement under service

$$I_t = \frac{1}{3} \cdot (b) (k \cdot d_e)^3 + n \cdot A_{s\_cap} \cdot (d_e - k \cdot d_e)^2$$

$$y = d_e - k \cdot d_e$$

$$f_s = \frac{\left(Mu_{cap\_ser1} \cdot 12 \frac{in}{ft} \cdot y\right)}{I_t} \cdot n$$

Pile Cap - Pile Joint Design Check AASHTO Seismic 8.13

f'c=	5.20	ksi			expected concrete strength
Bcap=	48.00	in			width of pile cap
Ds=	72.00	in			depth of pile cap
Dc=	24.00	in			pile diameter
Beff=	33.94	in			AASHTO SEISMIC 8.13.2-9
Ast=	16.00	in2			8 bundles of #9
Fye=	68.00	ksi			expected rebar strength
Tc=	761.60	kips			AASHTO Seismic 8.13.2-8
lac=	66.00	in			5.5' ℓ_{ac} = length of column reinforcement embedded into the bent cap (in.)
Vjv=	0.34	ksi			AASHTO SEISMIC 8.13.2-7
Pc=	416.00	kips			Max axial force in Pile during EXT event
fv=	0.09	ksi			AASHTO SEISMIC 8.13.2-6
fh=	0.00	ksi			No axial force in pile cap
			Allowable		
pt=	0.30		0.87	ksi	AASHTO SEISMIC 8.13.2-2 & AASHTO SEISMIC 8.13.2-3
pc=	0.39		1.3	ksi	AASHTO SEISMIC 8.13.2-1 & AASHTO SEISMIC 8.13.2-4

Calculated Stress

Hence, calculated stress are less than allowable stress, joint sizing is ok

Minimum Joint Shear Reinforcing AASHTO SEISMIC 8.13.3 (In Pile)

Required=	0.15%		based on AASHTO SEISMIC 8.13.3-2 since, pt > (0.11 x sqrt f'c)
Asp=	0.2	in2	#4 spiral
s=	6	in	spacing in cap
D'=	22.75	in	plug diameter
provided=	0.59%		#4 spiral at 6" SPA OK

$$\rho_s = \frac{4A_{sp}}{sD'}$$

$$\rho_s \geq 0.40\frac{A_{st}}{\ell_{ac}^2} \quad (8.13.3\text{-}2)$$

where:

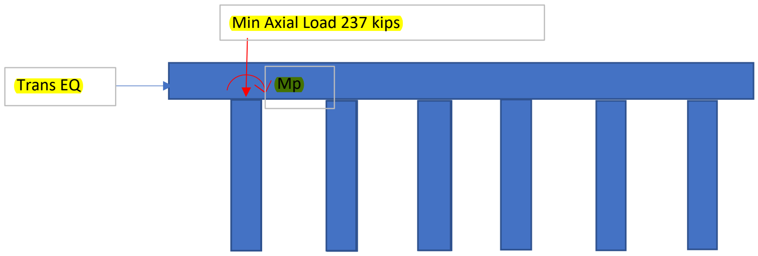
A_{st} = total area of column reinforcement anchored in the joint (in.^{2})

ℓ_{ac} = length of column reinforcement embedded into the bent cap (in.)

Minimum Joint Shear Reinforcing AASHTO SEISMIC 8.13.5 checked using STM; see next page

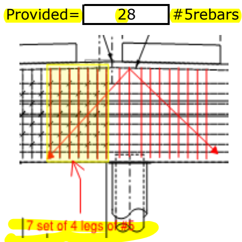
To Check whether shear reinforcement in cap beam is adequate to transfer Mp from Pile to Pile Cap during EQ

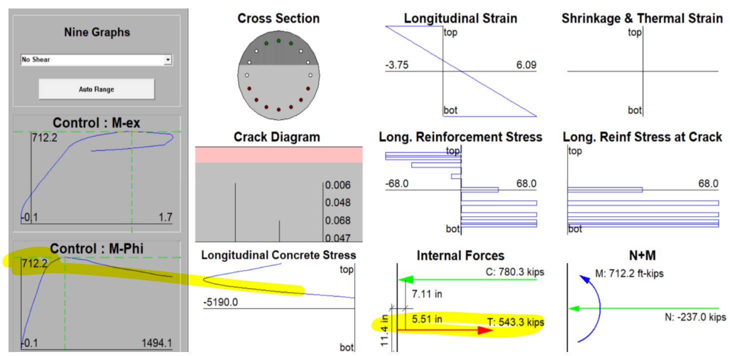
User Input= | References | Notes



Moment Mpo Trans Direction from pile to beam:

T, (response2000)=	543	kips
f'y=	68	ksi
Φ=	1	
Rebar area required, A =	8.0	in2
Needed=	26	#5rebars
Provided=	28	#5rebars





COWI

PROJECT:	I-405 R2B Final Design - BR 28 W	JOB NO.:	A207833
SUBJECT:	Pile Cap Joint Design - Abutment 1 West		
BY:	RSGR	DATE:	9/13/2021
CHK:	MEDN	DATE:	9/13/2021

Pile Cap - Pile Joint Design Check AASHTO Seismic 8.13

	f'c=	5.20	ksi			expected concrete strength
	Bcap=	48.00	in			width of pile cap
	Ds=	72.00	in			depth of pile cap
	Dc=	24.00	in			pile diameter
	Beff=	33.94	in			AASHTO SEISMIC 8.13.2-9
	Ast=	24.96	in2			8 bundles of #11
	Fye=	68.00	ksi			expected rebar strength
	Tc=	1188.10	kips			AASHTO Seismic 8.13.2-8
	lac=	66.00	in			5.5' ℓ_{ac} = length of column reinforcement embedded into the bent cap (in.)
	Vjv=	0.53	ksi			AASHTO SEISMIC 8.13.2-7
	Pc=	351.00	kips			Max axial force in Pile during EXT event
	fv=	0.08	ksi			AASHTO SEISMIC 8.13.2-6
	fh=	0.00	ksi			No axial force in pile cap
				Allowable		
Calculated Stress	pt=	0.49		0.87	ksi	AASHTO SEISMIC 8.13.2-2 & AASHTO SEISMIC 8.13.2-3
	pc=	0.57		1.3	ksi	AASHTO SEISMIC 8.13.2-1 & AASHTO SEISMIC 8.13.2-4

Hence, calculated stress are less than allowable stress, joint sizing is ok

Minimum Joint Shear Reinforcing AASHTO SEISMIC 8.13.3 (In Pile)

Required=	0.23%		based on AASHTO SEISMIC 8.13.3-2 since, pt > (0.11 x sqrt f'c)
Asp=	0.2	in2	#4 spiral
s=	6	in	spacing in cap
D'=	22.75	in	plug diameter
provided=	0.59%		#4 spiral at 6" SPA OK

$$\rho_s = \frac{4A_{sp}}{sD'}$$

$$\rho_s \geq 0.40\frac{A_{st}}{\ell_{ac}^2} \quad (8.13.3\text{-}2)$$

where:

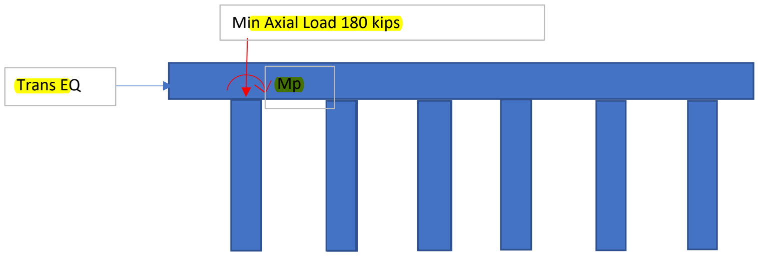
A_{st} = total area of column reinforcement anchored in the joint (in.^{2})

ℓ_{ac} = length of column reinforcement embedded into the bent cap (in.)

Minimum Joint Shear Reinforcing AASHTO SEISMIC 8.13.5 checked using STM; see next page

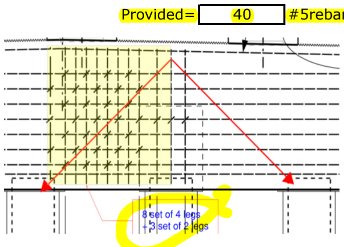
To Check whether shear reinforcement in cap beam is adequate to transfer Mp from Pile to Pile Cap during EQ

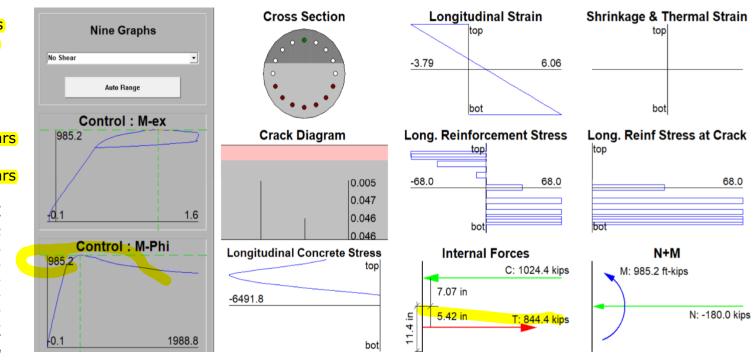
User Input= References Notes



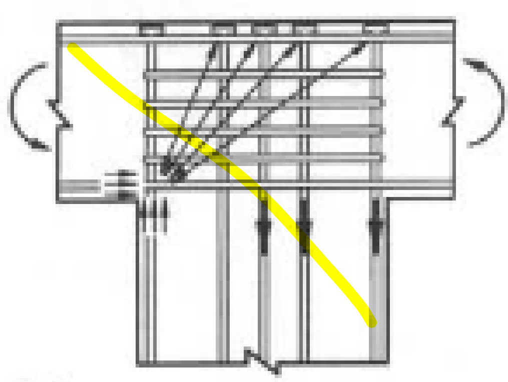
Moment Mpo Trans Direction from pile to beam:

T, (response2000)=	844	kips
f'y=	68	ksi
Φ=	1	
Rebar area required, A =	12.4	in2
Needed=	40	#5rebars
Provided=	40	#5rebars





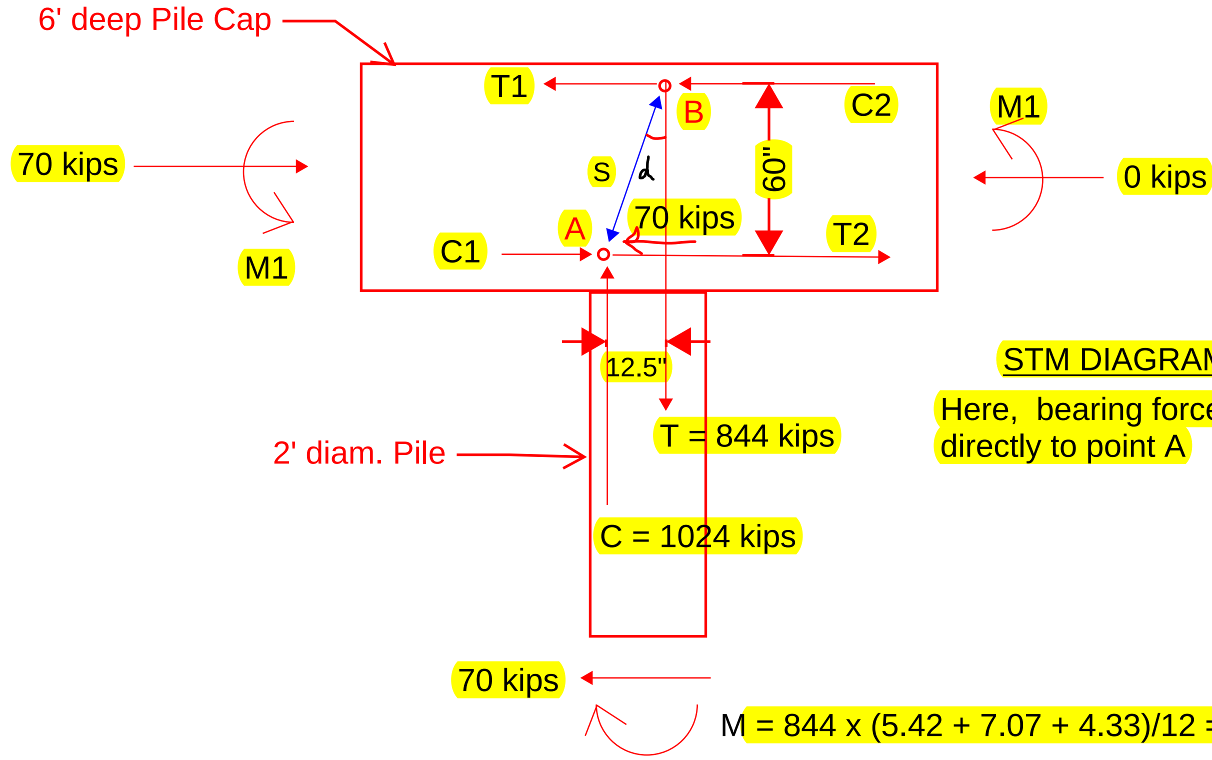
Complete STM model on next 2 pages to transfer pile moment into pile cap based on the following STM model for headed bars from (Priestley, et al., Seismic Design and Retrofit of Bridges)



(c) Headed Column Longitudinal Bars

FIG. 5.67 Possible mechanisms for tee-joint force transfer.

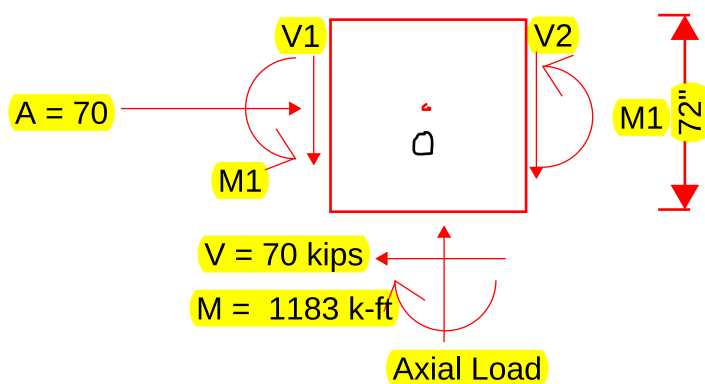
Complete STM to show transfer of pile forces to pile cap (MIDDLE PILE CASE)



M = 844 x (5.42 + 7.07 + 4.33)/12 = 1183 k-ft

STM DIAGRAM

Here, bearing force Br will strut directly to point A



Joint Equilibrium

Here, Axial load = V1 + V2. Assuming the equilibrium axial load is ignored in the STM diagram above and hence C=T=844 kips

M + V x (72"/2) = 2M1 (equilibrium about point O)

M1 = 696 k-ft

α = 11.77 degree

S = 844 / cos α = 862 kips

C1 = T1 + 70kips

C2 = T2 = M1/60" = 140 kips

C1 + T2 - 70 - S sin α = 0 =>

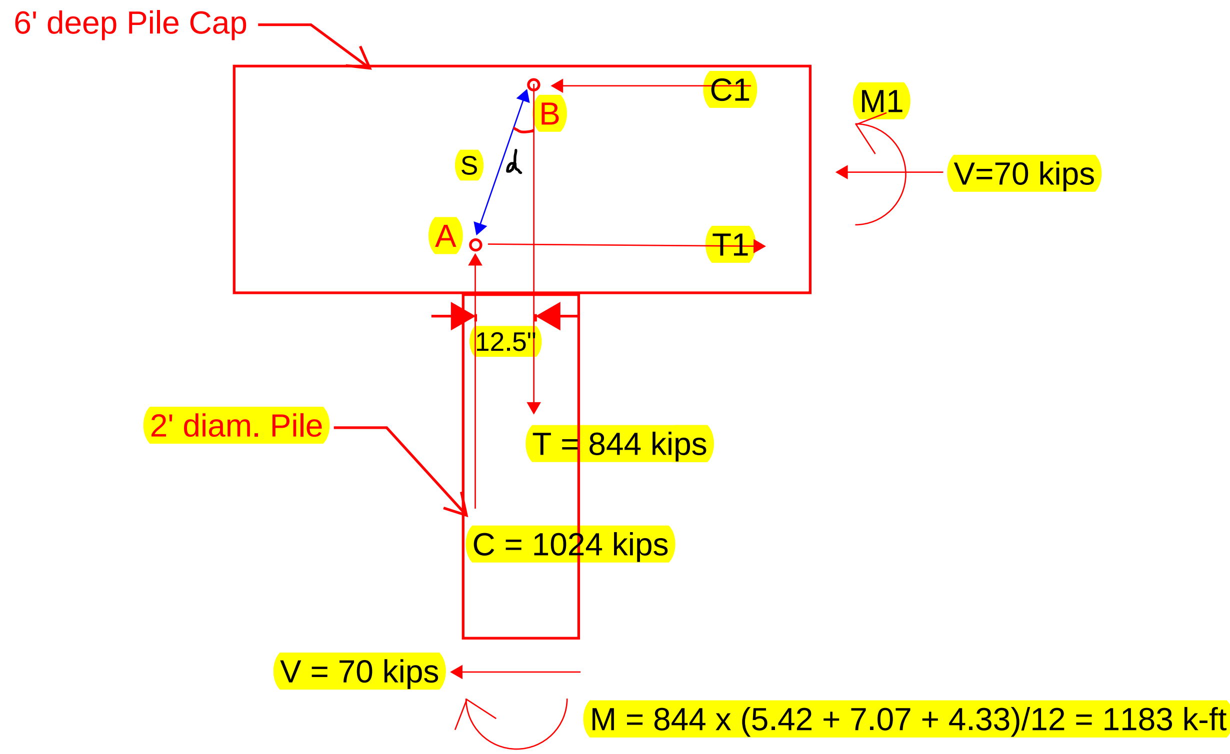
T1 + C2 - S sin α = 0;

T1 = 106-70 = 36 kips

C1 = 106 kips

T2 = C2 = 140 kips

Complete STM to show transfer of pile forces to pile cap (END PILE CASE)



α = 11.77 degree

S = 844 / cos α = 862 kips

T1 = C1 + 70kips

M + V x (72"/2) = M1

M1 = 1393 k-ft

C1 = M1/ 60" = 1393/60" = 279 kips
T1 = 279 + 70 = 349 kips

The pile cap main reinforcement has been designed with 5nos. #11 rebar due to plastic over strength moment in piles. + 2 nos. #11 rebars on corner. Total 7 nos. #11 rebar in each top and bottom. It is sufficient to resist the tension forces generated during pile to pile cap load transfer.

Fin Wall Design

PROJECT:	I-405 R2B Final Design - BR 28 W	JOB NO.:	A207833
SUBJECT:	Fin Wall Reinforcement Design		
BY:	RSGR	DATE:	9/13/2021
CHK:	MEDN	DATE:	9/13/2021

SUMMARY

	bar	Nos.	spacing	D/C
Main horizontal reinforcement	11	7	6.00	86%
Main top reinforcement	11	5	EQ	64%
Shear reinforcement	6	19	6.00	74%

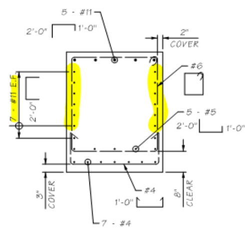
λ =	1.00		
Exposure Level =	Severe		
Strain Gradiant Amplification Factor β =	1.20		
γ =	1.50		
f'_{ce} =	5.20	ksi	*Expected property for extreme case in flexure
w_c =	0.155	kcf	
β_1 =	0.79		
Shear Reinforcement?	Yes		
Shear Reinforcement Size	6		
Shear Reinforcement Spacing	6.00	in	
# of Shear Reinforcement Legs	2.00		
Clear Cover (c) =	2.00	in	on sides
f_{ye} =	68.00	ksi	*Expected property for extreme case in flexure
ε_{ty} ($\varepsilon_{balanced}$) =	0.002	in/in	
$\varepsilon_{tension\ controlled}$ =	0.005	in/in	
ρ_{min} =	0.003		
ρ_{max} =	0.022		

Designed Main Horizontal bars:

	bar	11	#	
bar diam	dbi=	1.41	[in]	
bar area	Ab=	1.56	[in^2]	
	Depth of section, H=	36	[in]	
	Effective Depth to main bar, d=	33.295	[in]	=H-c-0.5*dbi-dbo
	Design width of section, bw =	38	in	=3'-8" - 6" bottom
	spacing of main bar, s=	6	[in]	
	Area of steel in design width, As=	10.92	[in^2]	=(bw/s)*Ab
		I		
	Resistance factor, φ=	1	EXT Case	
	depth of equiv stress block, a=	3.76	[in]	
	Factored Moment resistance, Mn=	1944	k-ft	

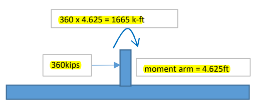
$$a = \frac{A_s \cdot f_y}{\phi \cdot f_c' \cdot b_w}$$

$$M_n = \phi \cdot A_s \cdot f_y \left(d - \frac{a}{2}\right)$$



Moment Demand: based on Fin wall taking entire transverse seismic load

Mu=	1665	k-ft	=360 kips is maximum possible Fin Wall load x (0.5 x fin wall length)

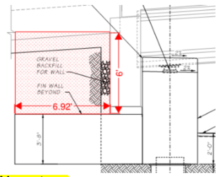


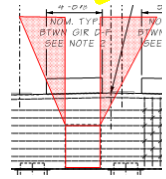
*conservative to use point load, actual load is distributed

COWI	PROJECT: I-405 R2B Final Design - BR 28 W		JOB NO.: A207833
	SUBJECT: Fin Wall Reinforcement Design		
	BY: RSGR		DATE: 9/13/2021
	CHK: MEDN		DATE: 9/13/2021

Designed Main Top bars:

design governed by STR case





Area of soil in Long Direction	41.5	ft2	=6.92 x 6				
Depth of wall	3	ft					
Volume	124.6	ft3					
Additional Volume	144	ft3	on the sides				
unit weight of soil	0.135	kcf					
Soil Weight	36.2	kips					
			Factor	Factored Load	Moment arm		
Weight of Fin Wall	15.8	kips	1.25	19.73	4.625	ft	
Soil Load	36.2	kips	1.3	47.07	5.8	ft	
Net downdrag on Fin wall	58	kips	1.4	81.20	5.8	ft	
Live Load (assume 2ft of soil height - 260pcf)	5.3976	kips	1.75	9.45	5.8	ft	
			Total, Vertical Load	157	kips		
			Total Moment, M=	890	k-ft		

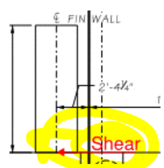
Designed Main Top bars: (use nominal material properties, fy = 60ksi and f'c = 4ksi)

	bar	11	#	
bar diam	dbi=	1.41	[in]	
bar area	Ab=	1.56	[in^2]	
	Depth of section, H=	44	[in]	
	Effective Depth to main bar, d=	41.295	[in]	=H-c-0.5*dbi-dbo
	Design width of section, bw =	36	in	
	Area of steel in design width, As=	7.8	[in^2]	=(bw/s)*Ab
	Resistance factor, φ=	0.9	EXT Case	
	depth of equiv stress block, a=	3.25	[in]	
	Factored Moment resistance, Mn=	1392	k-ft	

$$a = \frac{A_s \cdot f_y}{\phi \cdot f'_c \cdot b_w}$$

$$M_n = \phi \cdot A_s \cdot f_y \left(d - \frac{a}{2} \right)$$

SHEAR (f'c = 4ksi; fy = 60ksi)



Max Fin Wall lateral load=	360	kips					
Wall length=	9.25	ft					
Distributed Load=	38.92	k/ft					
Effective dept, dv=	33.9	in					AASHTO 5.7.2.8
Shear at dv away from pile cap, Vu	250.2	kips					
tensile strain in section, ε_s=	0.003	in/in					AASHTO 5.7.3.4.2-4
crack spacing parameter (1), sx=	6	in					
crack spacing parameter (1), sxe=	6	in					
shear resistance parameter, β=	1.66						AASHTO 5.7.3.4.2-2
Concrete modification factor, λ=	1						AASHTO 5.4.2.8
Nominal Shear Resistance, Vc=	128	kips					AASHTO 5.7.3.3-3
Factored Shear Resistance, Vr=φVc	115	kips	NG Add Transverse Steel				
bar size=	6						
spacing,s=	6	in					
Smax=	20	in	Prov spacing is OK				=0.6xdv
bar diam=	0.75	in					
Θ=	39.6						AASHTO 5.7.3.4.2-4
Legs=	2.0						
Length in which shear is distributed based on Θ=	28.0	in					
Steel area, Av=	4.1	in2					
				LIMIT			
Shear resistance from steel, Vs=	247	kips					AASHTO 5.7.3.3-4
Factored Shear Resistance, φVn=φ(Vc+Vs)	337	kips/ft	OK	1015	kips/ft	OK	Limit AASHTO 5.7.3.3-2

Curtain Wall Design

The curtain walls are designed for full mobilization of passive pressure For abutment 2 only

SUMMARY

	bar	Nos.	spacing	D/C
Main vertical reinforcement	8	14	6.00	99%
Main horizontal reinforcement	8	20	6.00	63%

λ =	1.00		
Exposure Level =	Severe		
Strain Gradiant Amplification Factor β =	1.20		
γ =	1.50		
f'_c =	5.20	ksi	*Expected property for extreme case
w_c =	0.155	kcf	
β_1 =	0.79		
Shear Reinforcement?	No		
Shear Reinforcement Size	#REF!		
Shear Reinforcement Spacing	0.00	in	
# of Shear Reinforcement Legs	0.00		
Clear Cover (c) =	3.00	in	
f_y =	68.00	ksi	*Expected property for extreme case
ε_{ty} ($\varepsilon_{balanced}$) =	0.002	in/in	
$\varepsilon_{tension\ controlled}$ =	0.005	in/in	
ρ_{min} =	0.003		
ρ_{max} =	0.022		

Designed Main vertical bars:

bar	8	#	
bar diam dbo=	1.00	[in]	
bar area Ab=	0.79	[in^2]	
Depth of section, H=	12	[in]	
Effective Depth to main bar, d=	8.5	[in]	=H-c-0.5*dbo
Design width of section, bw =	12	in	=check resistance for 1ft of wall
spacing of main bar, s=	6	[in]	
Area of steel in design width, As=	1.58	[in^2]	=(bw/s)*Ab
Resistance factor, φ=	1	EXT Case	
depth of equiv stress block, a=	1.72179	[in]	
Factored Moment resistance, Mn=	68	k-ft/ft	
Moment resistance for 4ft of connection with pile cap=	**274**	k-ft	

$$a = \frac{A_s \cdot f_y}{\phi \cdot f'_c \cdot b_w}$$

$$M_n = \phi \cdot A_s \cdot f_y \left(d - \frac{a}{2}\right)$$

Moment Demand:

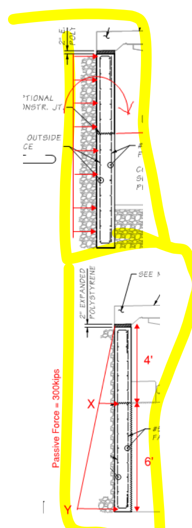
Comes from passive force being developed
Max Passive force developed at Abutment 2 (see pile load calculations)

max passive force developed in curtain wall, P=	347	kips	This is obtained from Pile Design Sheet
wall height, H=	10.24	ft	
wall length, L=	7	ft	
UDL, w=	34	kips/ft	=P/H
length of cantilever, L=	4	ft	
Moment Demand, Mu=	**271**	kips-ft	=w*L*L/2

Hence resistance of 274 k-ft is ok

Check for distribution triangularly

Y=	0.95	kcf	=P/(0.5 x 10.24 x 10.24 x 7)
X=	0.37	kcf	linear interpolation
Pressure Force in cantilever = F=	2.6	ksf	=X x L
Max. cantilevered moment arm= A=	4	ft	
Total Moment, M =	21	k-ft/ft	=F*A*A/2
Factored moment resistance for 1ft of wall=	68	k-ft/ft	OK>M



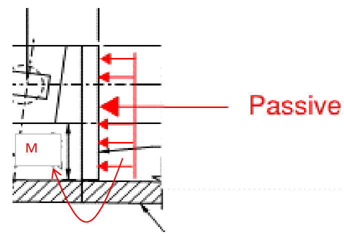
Designed Main Earth Face horizontal bars:

	bar	8	#
bar diam	dbi=	1.00	[in]
bar area	Ab=	0.79	[in^2]

Depth of section, H=	12	[in]	
Effective Depth to main bar, d=	7.5	[in]	=H-c-0.5*dbi-dbo
Design width of section, bw =	12	in	
spacing of main bar, s=	6	[in]	
Area of steel in design width, As=	1.58	[in^2]	=(bw/s)*Ab
Resistance factor, φ=	1	EXT Case	
depth of equiv stress block, a=	1.72	[in]	
Factored Moment resistance, Mn=	59	k-ft/ft	

$$a = \frac{A_s \cdot f_y}{\phi \cdot f'_c \cdot b_w}$$

$$M_n = \phi \cdot A_s \cdot f_y \left(d - \frac{a}{2}\right)$$



Moment Demand:

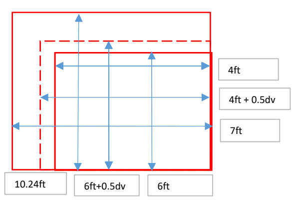
Comes from passive force being developed
Max Passive force developed at Abutment 2 (see pile load calculations)

Max passive force on cantilever=	347	kips	
Length of wall over which this is distributed=	7	ft	
UDL, w=	49.6	kips/ft	=347/7
length of cantilever, L=	3.00	ft	
Total Moment, M =	223	k-ft	=w*L*L/2

Factored moment resistance for 1ft of wall=	59	k-ft/ft	
Factored moment resistance for 6ft of wall=	357	k-ft	**OK**

Check punching:

Total Load on curtain wall=	347	kips
Resisted by pile cap (based on area)=	116	kips
Punching Shear ,V=	231	kips



dv=	8.64	in	
bo=	128.64	in	=perimeter of critical section as shown above in dotted lines
β=	1.5	=6 / 4	=ratio of long side to short side of pile cap
λ=	1		
f'c=	4	ksi	
Vn=	327	kips	
0.9 factored Vn=	294	kips	
	Hence, punching shear is ok		
D/C=	78%		

$$V_n = \left(0.063 + \frac{0.126}{\beta_c}\right)\lambda\sqrt{f'_c}\, b_o d_v \le 0.126\lambda\sqrt{f'_c}\, b_o d_v \quad (5.12.8.6.3\text{-}1)$$

Check One Way Shear

Total Load on curtain wall=	347	kips
shear distributed along horizontal=	87	kips
shear distributed along vertical=	58	kips

Along horizontal

$$V_c = 0.0316\beta\lambda\sqrt{f'_c}\; b_v d_v \quad (5.7.3.3\text{-}3)$$

bv=	81.996	in		
dv=	8.64	in		
Vc=	90	kips	D/C=	97%

Along horizontal

bv=	122.004	in		
dv=	8.64	in		
Vc=	133	kips	D/C=	43%

Abutment Seat Length Check

COWI	PROJECT	I-405 BR 28W		CONT	A207833
	SUBJECT	Summary of Abutment Seat Length C-D Ratios		PAGE	-
CHECKED BY MWBM	DATE	2021-01-14	CALCS BY RSGR	DATE	2021-01-13

SUMMARY ABUTMENT SEAT LENGTH-CAPACITY / DEMAND RATIO

LONGITUDINAL DIRECTION					
Abutment	C, pre-lim	D	C/D	C, provide	C/D
	in	in	Unitless	in	Unitless
West	48	36.00	1.33	48	1.33
East	48	36.00	1.33	48	1.33

: User Input

METHODOLOGY:

• As the large longitudinal movements may cause unseating of the girder and resulting in the collapse of the bridge the seat length demands were determined as per FHWA Eq. 4-3. The acceleration coefficient is determined form Bridge Link. This value was compared to the seat length demand from AASHTO seismic and the maximum value was considered as demand.

• The existing capacity is being designed, C

COWI		PROJECT	I-405 BR 28W		CONT	A207833
		SUBJECT	Abutment Seat Length Demand Calculation		PAGE	-
CHECKED BY	MWBM	DATE	2021-01-14	CALCS BY RSGR	DATE	2021-01-13

ABUTMENT SEAT LENGTH DEMAND

West:

User input=			
Length of bridge deck from seat to end of bridge deck, L=	116.5	ft	
Average height of columns supporting the bridge Deck , H=	0	ft	
Width of the deck, B=	50.67	ft	
1-Sec Period Acceleration Coefficient, S_{D1} =	0.283		
Angle of Skew, α =	0.0000	deg	
Seismic Displacement demand Δ_{EQ} =	5.0000	in	Assumption
Support Length measured normal to the face of abutment, N_{fhwa} =	8.57	in	FHWA Eq. 4-3b
Minimum support length measured normal to the centerline of brg, N=	24.00	in	AASHTO Seismic Eq. 4.12.3-1 (For SDC D)
% N by SDS and acceleration coefficient, for SDC:D =	150	%	AASHTO Seismic Table 4.12.2-1
N_{AASHTO} =	36.00	in	
N=	36.00	in	

East:

User input=			
Length of bridge deck from seat to end of bridge deck, L=	116.5	ft	
Average height of columns supporting the bridge Deck , H=	0	ft	
Width of the deck, B=	50.67	ft	
1-Sec Period Acceleration Coefficient, S_{D1} =	0.283		
Angle of Skew, α =	0.0000	deg	
Seismic Displacement demand Δ_{EQ} =	5.0000	in	Assumption
Support Length measured normal to the face of abutment, N_{fhwa} =	8.57	in	FHWA Eq. 4-3b
Minimum support length measured normal to the centerline of brg, N=	24.00	in	AASHTO Seismic Eq. 4.12.3-1 (For SDC D)
% N by SDS and acceleration coefficient, for SDC:D =	150	%	AASHTO Seismic Table 4.12.2-1
N_{AASHTO} =	36.00	in	
N=	36.00	in	

Girder Stop Design

COWI

PROJECT i405 R2B CONT

SUBJECT GIRDER STOP B28 PAGE 01

CHECKED BY DATE CALCULATIONS BY DATE

DESIGN LOAD.

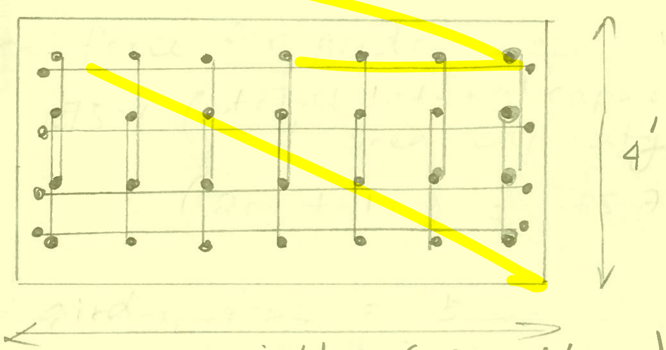
Total Lateral EQ Load = Mass of Structure x F x PGA = (1228+1250)kips x 1.17 x 0.43 = 1246kips

Assume unequal distribution by assuming only 50% of girder stops are effective
Hence, multiply total lateral load by 2 = 1246 x 2 = 2492 kips

Total girder stops = 5 on the west and 5 on the east = 10nos.

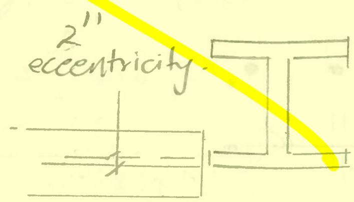
Load per girder stop = 2492 / 10 = 249 kips

DESIGNED CONFIGURATION

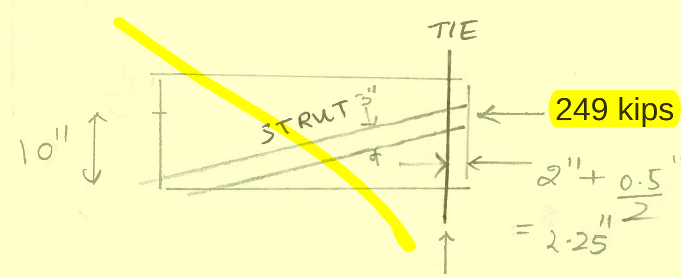


	Height (in)
Pier 1	16"
Pier 2	15"

FORCE POINT OF APPLICATION



SIMPLIFIED STM MODEL



STRUT WIDTH = 3" (assumed).

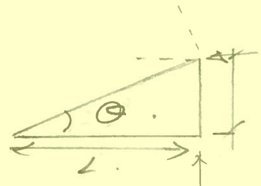
COWI

PROJECT ______ CONT ______

SUBJECT ______ PAGE 02

CHECKED BY ______ DATE ______ CALCULATIONS BY ______ DATE ______

STM check



max Tie force = 125kips when θ = 30 deg (min. angle 25 degree as per AASHTO 5.8.2.2)

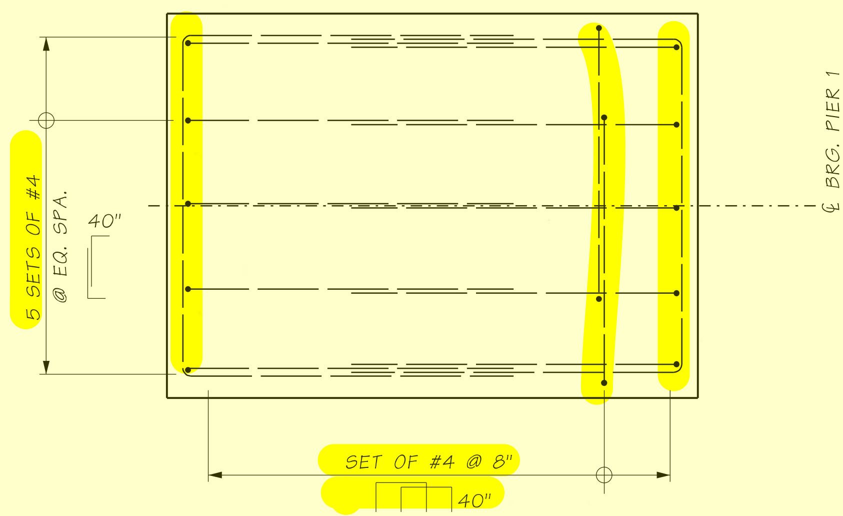
L @ θ = 30 deg = 17"

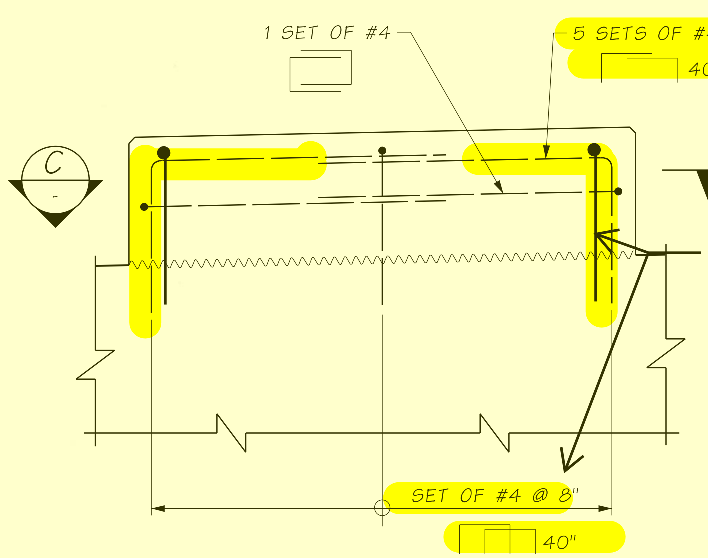
atleast 2 rows of bars present within 17"

$$\text{Area req.} = \frac{125 \text{ kips}}{0.9 \times Fye} = \frac{125}{0.9 \times 68 \text{ ksi}} = 2 \text{ in}^2$$

provided at least 9 nos. of # 4 bar in 1 row.

$A_{prov} = 1.8 \text{ in}^2$. < required 2 in^2 but considered ok since we conservatively assume 50% are effective





1 set of #4 at corner containing 4 dowels

COWI

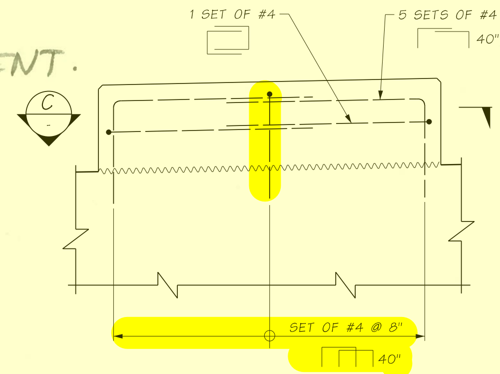
PROJECT ________ CONT ________

SUBJECT ________ PAGE 03

CHECKED BY ________ DATE ________ CALCULATIONS BY ________ DATE ________

Check using shear friction theory.
(Interface shear) AASHTO 5.7.4

* GIRDER STOP IS CAST AFTER GIRDERS ARE PLACED ON ABUTMENT.



1. min. interface reinforcement

$$A_{vf} \geq \frac{0.05\, A_{cv}}{f_y}$$

A_{cv} = interface concrete area
lets' say min. width = 4'
area = 48" × 12" = 576 in^2/ft (length of stop is variable)

$$A_{vf} \geq \frac{0.05 \times 576}{f_y} = 0.48\ in^2/ft.$$

provide atleast 4 dowels of #4 rebar in 1ft length. A_{vf} = 0.8 in^2/ft.

2. Factored shear resistance = ϕV_{ni}
$\phi = 0.9$ WSDOT 7.5.6 (c)

$$V_{ni} = cA_{cv} + \mu (A_{vf} f_y)$$

c^{**} = 0.24 ksi
μ = 1.0
Roughened surface

V_{ni} = 1.0 × 0.6 in^2/ft × 60 + C (Acv) = 36 + 0.24*(576) = 174

factored resistance ϕV_{ni} = 0.9 × 174 = 157 kips/ft

All girder stop lengths in B28W are atleast 4' long.
Hence, ϕV_{ni} = 157 × 4 = 626 kips > Design load (249 kips)

Check Pile Cap to Jacking forces

COWI	PROJECT: I-405 R2B Final Design - BR 28 W	JOB NO.: A207833	
	SUBJECT: Jacking Bearing Check		
	BY: RSGR	DATE:	4/11/2021
	CHK: MEDN	DATE:	4/11/2021

BEARING DESIGN TABLE AASHTO METHOD B DESIGN	
SERVICE - I LIMIT STATE	
DEAD LOAD (DL) REACTION	225 KIPS
LIVE LOAD REACTION (W/O IMPACT)	94 KIPS
UNLOADED HEIGHT	2.87 IN
LOADED HEIGHT (DL)	2.81 IN
SHEAR MODULUS	165 PSI

Max. DL	225	kips	
Jacking Load	450	kips	200% of DL

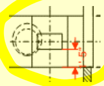
Jack used: CUSP250, 285.6 ton Capacity, 0.61 in Stroke, High Tonnage Ultra Flat Hydraulic Cylinder

Capacity	250	ton	
	551	kips	OK

Jack Size

Diam.	10	in
Height	3	in

Pier 1				Pier 2		
Available space	17	in	OK	17	in	OK



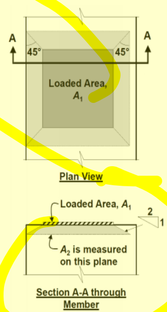


Figure 5.6.5-1—Determination of Notional Area

On a bearing plate min. 12"x12"

a=	12	in
c=	2	cover each side
a+2c=	16	in
A2=	256	=16 x 16
A1=	144	=12 x 12
m=	1.333333	=sqrt (A2/A1)
φ=	0.7	resistance factor
f'c=	4	ksi
A1=	144	in2
Pr=	457	kips OK

- Where the loaded area is subjected to uniformly distributed bearing stresses:

$$m = \sqrt{\frac{A_2}{A_1}} \leq 2.0 \qquad (5.6.5\text{-}3)$$

$$P_n = 0.85 f'_c A_1 m$$

P_n = nominal bearing resistance (kip)
A_1 = area under bearing device (in.^{2})
m = confinement modification factor
A_2 = notional area defined as shown in Figure 5.6.5-1 (in.^{2})

COWI

PROJECT: I-405 R2B Final Design - BR 28
SUBJECT: Pile Cap Design Check for Jacking Torsion Case
BY: RSGR
CHK: MEDN

JOB NO.: A207833
DATE: 4/15/2021
DATE: 4/15/2021

SUMMARY

	bar	Nos.	spacing	D/C
Main M- reinforcement at top	11	5	7.00	39%
Main M+ reinforcement at bottom	11	5	7.00	42%
Skin reinforcement each side	6		8.00	-
Transverse reinforcement (2 legs)	5	4	8.00	79%

			Unit	NOTES AND REFERENCES
λ =		1.00		AASHTO (5.4.2.8-2)
Exposure Level =		Severe		
Strain Gradiant Amplification Factor β =		1.20		
γ =		1.50		
f'_c =		4.00	ksi	
w_c =		0.155	kcf	
β_1 =		0.85		
Beam Depth, h =		72.00	in	
Beam Width, b_{wt}=		48.00	in	at top
Beam Width, b_{wb}=		48.00	in	at bottom
Shear Reinforcement?		Yes		
Shear Reinforcement Size	#	5	0.625	
Shear Reinforcement Spacing		8.00	in	
# of Shear Reinforcement Legs		4.00		
Clear Cover (c_c) =		6.00	in	
f_y =		60.00	ksi	
ε_{ty} ($\varepsilon_{balanced}$) =		0.00	in/in	AASHTO 5.6.2.1
$\varepsilon_{tension\ controlled}$ =		0.01	in/in	AASHTO 5.6.2.1
ρ_{min} =		0.00		
ρ_{max} =		0.02		

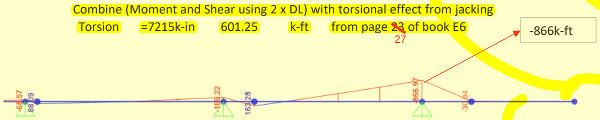
Designed Main bars:

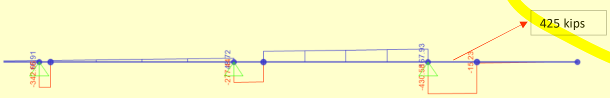
bar	11.00	
bar diam	1.41	in
bar area	1.56	in2

Demands (Taken at face of pile)

Combine (Moment and Shear using 2 x DL) with torsional effect from jacking

Torsion =7215k-in 601.25 k-ft from page ~~23~~ 27 of book E6





	STR Case		M (k-ft)	T (k-ft)	V (kips)
Max +Moment Case			866	601	425
Max -Moment Case			-866	601	425
Max Torsion Case			-866	601	425
Max Shear Case			-866	601	425

at Pile Face
from SAP

The effective is calculated as follows:

Cover top	2	in
Cover bottom	6	in
de top reinforcement	68.67	in
de bottom reinforcement	64.67	in

Solve for the required amount of reinforcing steel, as follows:

	Top	Unit			Bottom	Unit		Notes
φf=	0.9							AASHTO 5.5.4.2
b=	48.00	in						top
fc=	4	ksi						
Max -M =	866	k-ft		Max +M	866	k-ft		
Rn=	0.0510	ksi			0.0575	ksi		
ρ=	0.00086				0.00097			
As,req=	2.82380	in2			3.00140	in2		
Nos. prov	5.00000				5.00000			
Bar size	11.00000				11.00000			
Bar Area	1.56000	in2			1.56000	in2		
$A_{s,prov}$ =	7.80000	in2			7.80000	in2		
Spacing, s=	10.33500	in			8.33500	in		
Use	7.00000	in			7.00000	in		Spacing provided
a =	11.47059	in			11.47059	in		Depth of rectangular compression block
$\varphi_f M_n$ =	2209.0	k-ft	OK		2069	k-ft	OK	Flexural capacity
strain compatibility check c=	2.15917	in	OK		1.88927	in	OK	AASHTO 5.7.3.1.1-4; 5.7.2.1-1
tensile stress check et=	0.09241		OK		0.10604		OK	

$$Rn = \frac{Mu_{cap\_str1} \cdot 12 \frac{in}{ft}}{\left(\phi_f \cdot b \cdot d_e^2\right)}$$

$$\rho = 0.85\left(\frac{f_c}{f_y}\right) \cdot \left[1.0 - \sqrt{1.0 - \frac{(2 \cdot Rn)}{(0.85 \cdot f_c)}}\right]$$

COWI	PROJECT: I-405 R2B Final Design - BR 28		JOB NO.: A207833
	SUBJECT: Pile Cap Design Check for Jacking Torsion Case		
	BY: RSGR		DATE: 4/15/2021
	CHK: MEDN		DATE: 4/15/2021

The minimum reinforcement requirements will be calculated for the cap.

The cracking strength is calculated as follows:

fr=	0.48	ksi			AASHTO 5.4.2.6
Ig=	1492992.00	in4			
yt=	36.00	in			
γ1=	1.60				AASHTO 5.6.3.3
γ3=	0.67				AASHTO 5.6.3.3
Sc=	41472.00	in3			
Mcr=	1778.32	k-ft			AASHTO eq 5.6.3.3-1
$\varphi_f M_{n(-)}$ =	2209		>	1152	OK
$\varphi_f M_{n(+)}$ =	2069		>	1152	OK

$$f_r = 0.24\cdot\sqrt{f_c}$$

$$M_{cr} = \gamma_3\left[\left(\gamma_1 f_r + \gamma_2 f_{cpe}\right)S_c - M_{dnc}\left(\frac{S_c}{S_{nc}}-1\right)\right] \quad (5.6.3.3\text{-}1)$$

Design for Shear and Torsion

The presence of torsion affects the total required amount of both longitudinal and transverse reinforcing steel. However, if the applied torsion is less than one-quarter of the factored torsional cracking moment, then the Specifications allow the applied torsion to be ignored. This computation is shown as follows:

φt=	0.90			Resistance factor for shear and torsion
Area of concrete Acp=	3456.00	in2		
Perimeter Pc=	240.00	in		
K=	2.00			
Tcr=	2073.60	k-ft		AASHTO eq 5.7.2.1-4
0.25 x φt x Tcr=	466.56	k-ft	Need to Check for Torsion	Torsion Threshold AASHTO eq 5.7.2.1-3

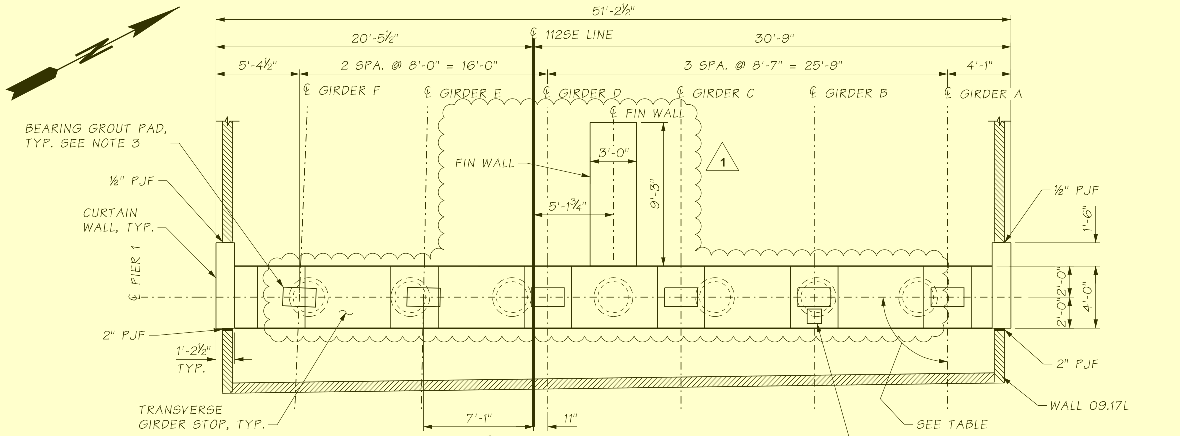
First Design the shear reinforcement and then check for torsional resistance
nominal shear resistance of the critical section is a combination of the nominal resistance of the concrete and the nominal resistance of the steel. This value is then compared to a computed upper-bound value and the lesser of the two controls. These calculations are illustrated below:

Shear bar size=	5			
Number of legs=	4			
Spacing of Shear bars, S=	8	in		
Clear Spacing=	7.375	in		
Av =	1.24	in^2		
dv=	62.93	in	effective shear depth	AASHTO C5.7.2.8-1
Use dv=	62.93	in		
β=	2.00			AASHTO 5.7.3.4.1
Theta=	45.00	deg		AASHTO 5.7.3.4.1
Use β =	1.62			AASHTO eq 5.7.3.4.2-1
Use Θ =	38.13			AASHTO eq 5.7.3.4.2-3
Vc=	309.95	kips		AASHTO eq 5.7.3.3-3
Shear Threshold=	139.48	kips		AASHTO eq 5.7.2.3-1
	Transverse Reinforcement Required			
A_v min =	0.40	in2		AASHTO eq 5.7.2.5-1
v_u =	0.16	ksi		Shear Stress AASHTO eq 5.7.2.8-1
s max =	24.00	in		AASHTO 5.7.2.6
e_s =	0.00			AASHTO eq 5.7.3.4.2-4
V_s =	745.62	kips		AASHTO eq 5.7.3.3-4
$\Phi V_n = V_r$ =	950.01	OK		

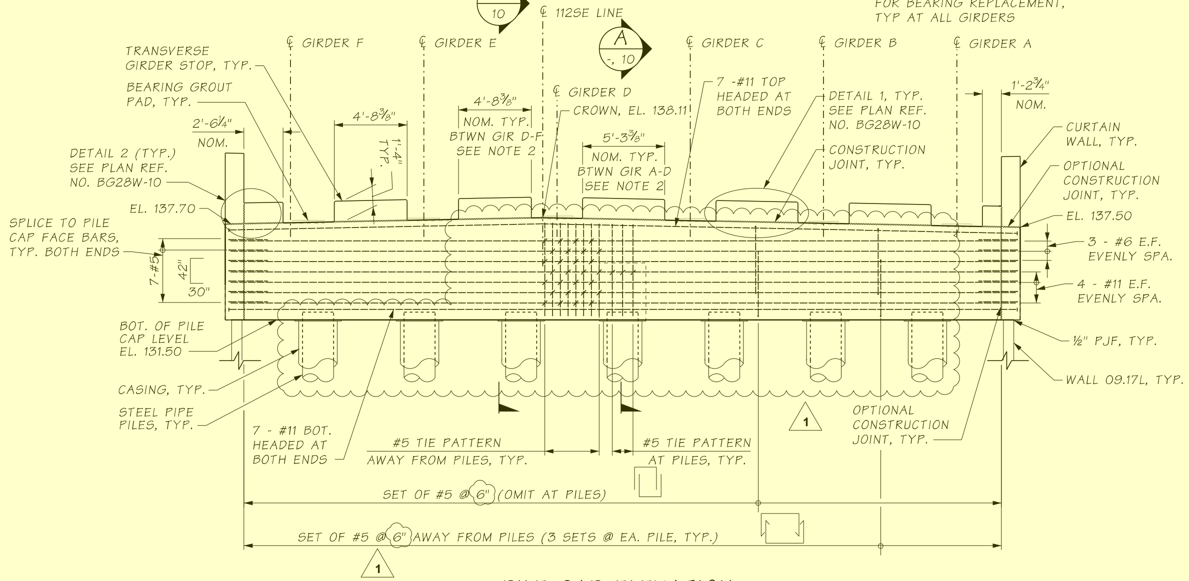
Check for torsional resistance

A_{cp} =	3456.00	in^2		
p_c =	240.00	in		
A_o =	2189.08	in^2	Area enclosed by shear path	85% of the hoop
p_h =	205.50	in		
Equivalent V_u =	522.92	kips		AASHTO eq 5.7.3.4.2-5
v_u =	0.19	ksi	shear stress	
s max =	24.00	in		
e_s =	0.00			AASHTO eq 5.7.3.4.2-4
β =	1.73			AASHTO eq 5.7.3.4.2-1
Θ =	37.30	degrees		AASHTO eq 5.7.3.4.2-3
V_c =	329.71	kips		AASHTO eq 5.7.3.3-3
V_s =	768.19	kips		AASHTO eq 5.7.3.3-4
$\Phi V_n = V_r$ =	988.11	kips		
A_t =	0.31	in^2		Area of transverse rebar
T_n =	1113.35	k-ft		AASHTO eq 5.7.3.6.2-1
$\Phi T_n = T_r$	1002.01	k-ft		
A_t / s - Req.	0.030	in^2 / in		
A_t / s - Prov.	0.039	in^2 / in	OK	D/C= 79%
Longitudinal A_s - Req.	9.71	in^2		AASHTO eq 5.7.3.6.3-1
A_s - Provided	15.60	in^2	OK	(7.8in^2 for 5nos. of #11 rebar) x 2 (top and bottom layer)
Add #11 bars at corner	6.24	in^2		Add corner bar for detailing at all 4 corners

Drawings



PLAN



PILE CAP ELEVATION

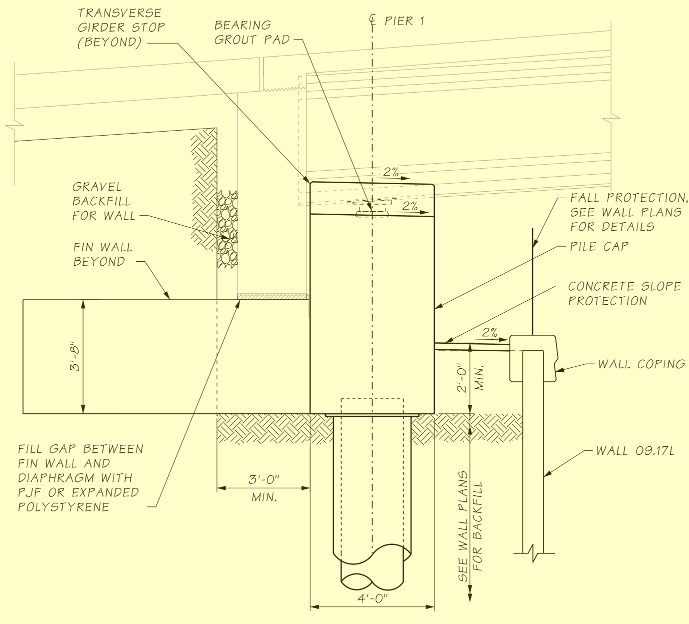
(LOOKING WEST)

DIMENSIONS AND ELEVATIONS ARE GIVEN ALONG ℄ PIER.
FIN WALL, GIRDER STOP AND CURTAIN WALL REINFORCEMENT NOT SHOWN, SEE PLAN REF. NO. BG28W-08 AND BG28W-10

NOTES:

1. FRONT SURFACE OF CONCRETE PILE CAP, GIRDER STOPS AND EXTERIOR FACE OF CURTAIN WALL SHALL RECEIVE "MT. ST. HELENS" PIGMENTED SEALER TO 1 FOOT BELOW FINISHED GRADE.
2. THE LENGTH OF TRANSVERSE GIRDER STOPS ARE NOMINAL. REFER TO PLAN REF. NO. BG28W-11 FOR DETAILS.
3. FOR DETAILS OF GROUT PAD RECESS, REFER TO PLAN REF. NO. BG28W-11.
4. FOR DETAILS OF WALL 09.17L, SEE OTHER PACKAGE.

TOP OF GROUT PAD	ELEVATION	ANGLE
GIRDER A	137.70	90°00'00"
GIRDER B	137.87	90°00'00"
GIRDER C	138.04	90°00'00"
GIRDER D	138.22	90°00'00"
GIRDER E	138.09	88°48'43"
GIRDER F	137.93	87°37'29"



SECTION A

FOR REINFORCEMENT DETAILS, SEE PLAN REF. NO. BG28W-10

FILE NO. 9242 SHEET

I-405

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Design Mgr:	BRIAN BELL	RELEASE FOR CONSTRUCTION RECORD			
Designed By:	R. GARG				
Checked By:	M. DASTFAN				
Detailed By:	K. BUNGER				
Current Revision By:		NDC-XX	09/09/2021	1	
Date:	9/14/2021	RFC - BRIDGE 405/28W	06/28/2021	0	
Time:	3:19:37 PM	DESCRIPTION	DATE	NO	

REGION NO.	STATE	FED. AID. PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			
JOB NUMBER XL5467				

MATTHEW D. BAUGHMAN, STATE OF WASHINGTON, 49664, STRUCTURAL PROFESSIONAL ENGINEER

E.O.R. 1 STAMP BOX DATE

E.O.R. 2 STAMP BOX DATE

Washington State Department of Transportation

FLATIRON LANE wood. COWI

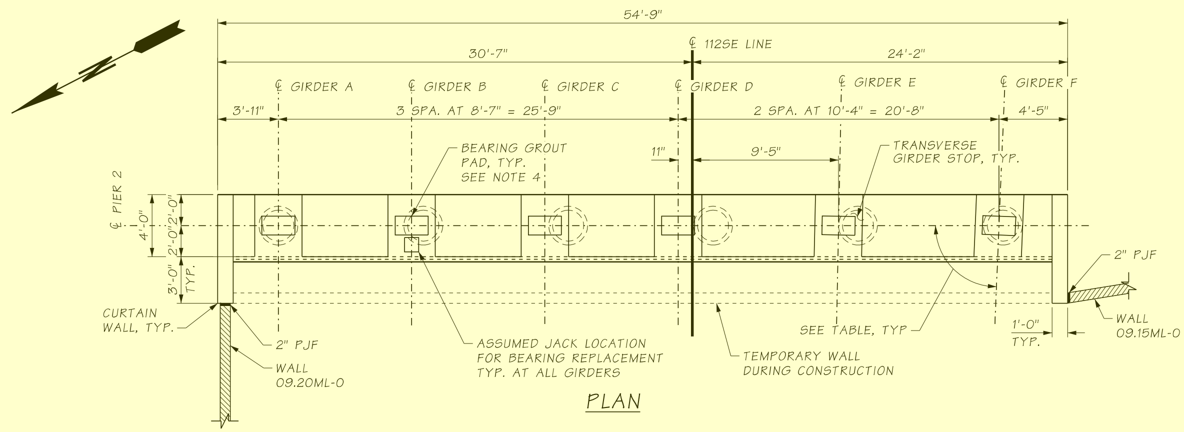
I-405; RENTON TO BELLEVUE WIDENING AND EXPRESS TOLL LANES PROJECT

112TH AVENUE SE OVER SB I-405

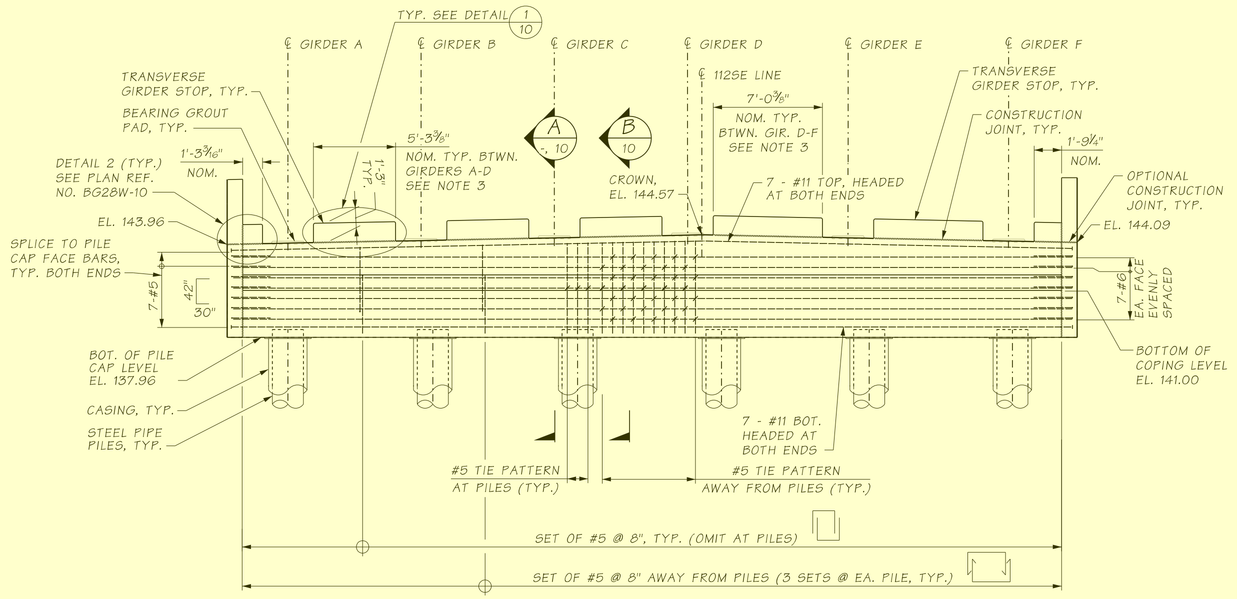
PIER 1 PLAN AND ELEVATION

PLAN REF. NO. BG28W-06

SHEET OF SHEETS



PLAN



PILE CAP ELEVATION

(LOOKING EAST)

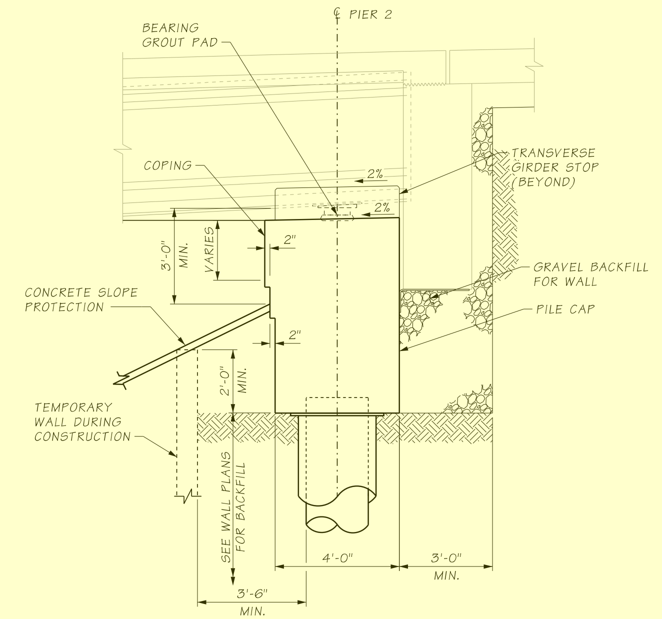
DIMENSIONS AND ELEVATIONS ARE GIVEN ALONG ℄ PIER

GIRDER STOP AND CURTAINWALL REINFORCEMENT NOT SHOWN, SEE PLAN REF. NO. BG28W-09 AND BG28W-10

NOTES:

1. FRONT SURFACE OF CONCRETE PILE CAP AND GIRDER STOPS SHALL RECEIVE "MT. ST. HELENS" PIGMENTED SEALER TO 1 FOOT BELOW FINISHED GRADE.
2. WALL COPING SHALL RECEIVE "MT. BAKER" PIGMENT SEALER.
3. THE LENGTH OF TRANSVERSE GIRDER STOPS ARE NOMINAL. REFER TO PLAN REF. NO. BG28W-11 FOR DETAILS.
4. FOR DETAILS OF GROUT PAD RECESS, REFER TO PLAN REF. NO. BG28W-11.
5. FOR DETAILS OF WALLS 09.15ML-0 AND 09.20ML-0, SEE OTHER PACKAGE.

TOP OF GROUT PAD	ELEVATION	ANGLE
GIRDER A	144.17	90°00'00"
GIRDER B	144.34	90°00'00"
GIRDER C	144.51	90°00'00"
GIRDER D	144.68	90°00'00"
GIRDER E	144.51	88°48'43"
GIRDER F	144.30	87°37'29"



SECTION A

FOR REINFORCEMENT DETAILS, SEE PLAN REF. NO. BG28W-10

I-405 FILE NO. 9242 SHEET

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Design Mgr:	STEVE WOODRUFF	RELEASE FOR CONSTRUCTION RECORD		
Designed By:	R. GARG			
Checked By:	M. DASTFAN			
Detailed By:	K. BUNGER			
Current Revision By:	P. CONRAD			
Date:	4/16/2021	RELEASE FOR CONSTRUCTION	04/16/2021	0
Time:	8:29:17 AM	DESCRIPTION	DATE	NO

REGION NO.	STATE	FED. AID. PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			

JOB NUMBER XL5467

MATTHEW D. BAUGHMAN · STATE OF WASHINGTON · 49664 · STRUCTURAL ENGINEER · PROFESSIONAL ENGINEER

E.O.R. 1 STAMP BOX DATE

E.O.R. 2 STAMP BOX DATE

Washington State Department of Transportation

FLATIRON LANE wood. COWI

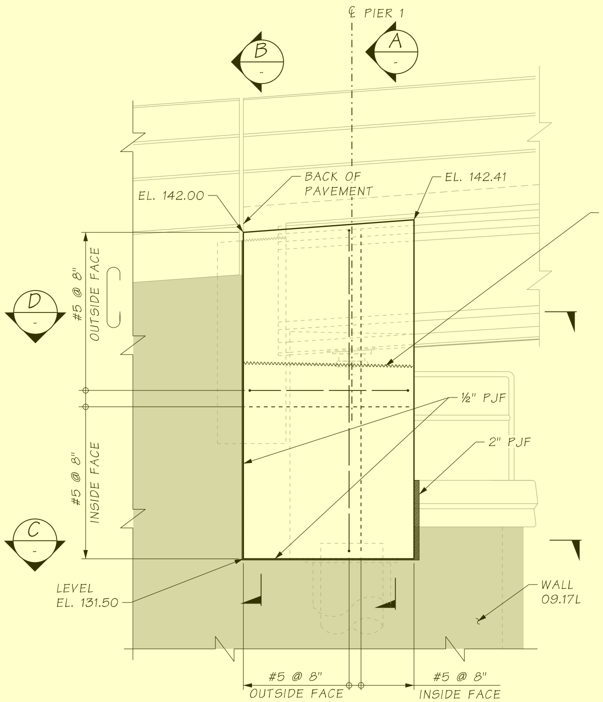
I-405; RENTON TO BELLEVUE WIDENING AND EXPRESS TOLL LANES PROJECT

112TH AVENUE SE OVER SB I-405

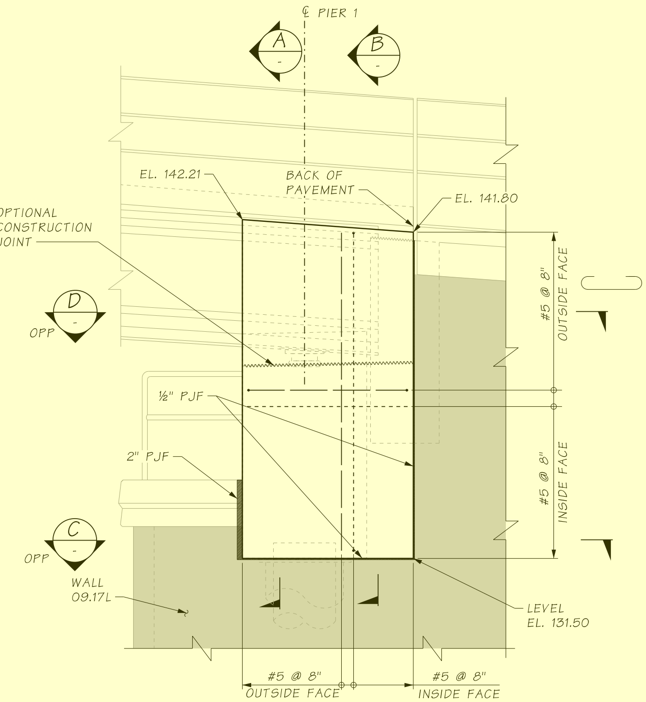
PIER 2 PLAN AND ELEVATION

PLAN REF. NO. BG28W-07

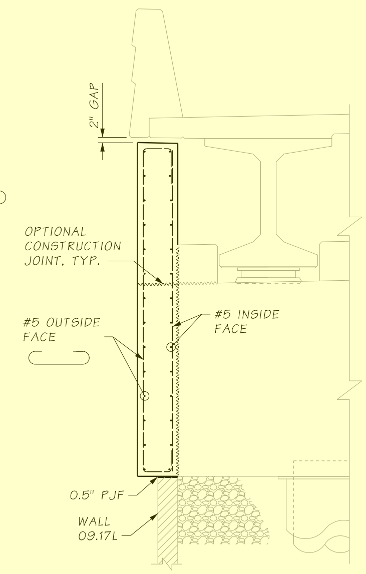
SHEET OF SHEETS



ELEVATION
PIER 1 - SOUTH CURTAIN WALL
(LOOKING NORTH)

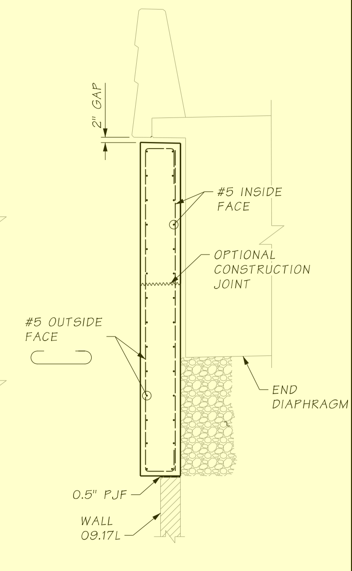


ELEVATION
PIER 1 - NORTH CURTAIN WALL
(LOOKING SOUTH)

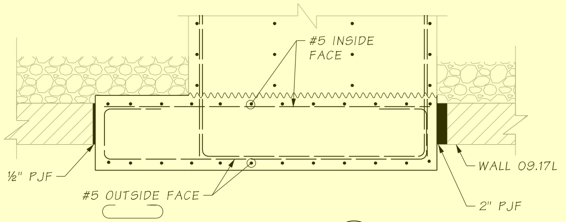


SECTION A

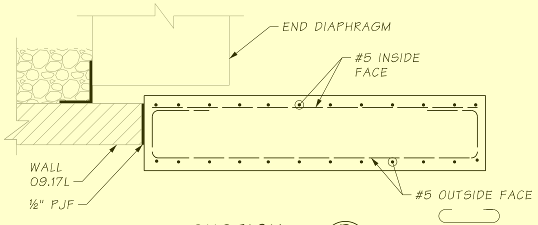
PIER CAP REINFORCEMENT NOT SHOWN



SECTION B



SECTION C



SECTION D

I-405 FILE NO. 9242 SHEET

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Design Mgr:	STEVE WOODRUFF	RELEASE FOR CONSTRUCTION RECORD		
Designed By:	M. DASTFAN			
Checked By:	M. BAUGHMAN			
Detailed By:	P. CONRAD			
Current Revision By:				
Date:	4/16/2021	RELEASE FOR CONSTRUCTION	04/16/2021	0
Time:	8:59:11 AM	DESCRIPTION	DATE	NO

REGION NO.	STATE	FED. AID. PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			

JOB NUMBER XL5467

MATTHEW D. BAUGHMAN STATE OF WASHINGTON 49664 PROFESSIONAL STRUCTURAL ENGINEER

E.O.R. 1 STAMP BOX DATE

E.O.R. 2 STAMP BOX DATE

Washington State Department of Transportation

FLATIRON LANE wood. COWI

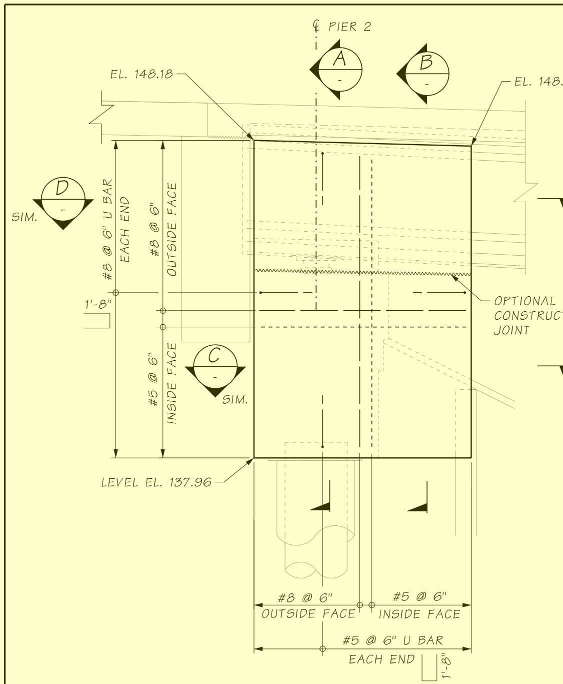
I-405; RENTON TO BELLEVUE WIDENING AND EXPRESS TOLL LANES PROJECT

112TH AVENUE SE OVER SB I-405

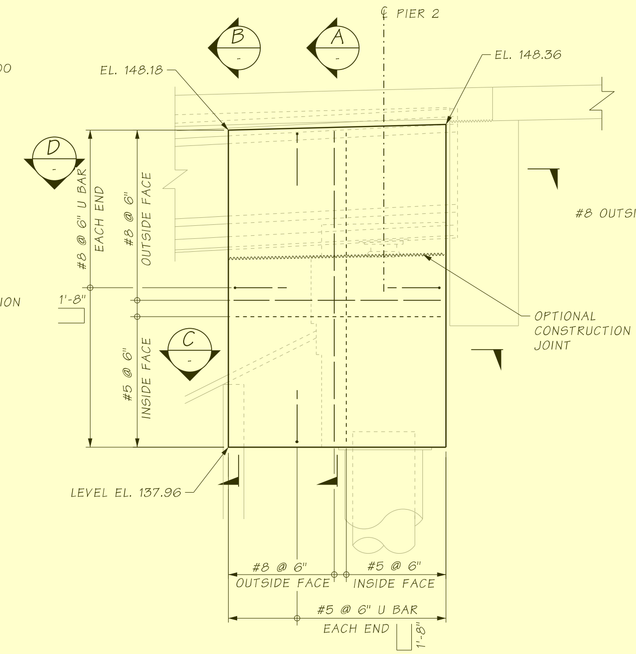
PIER DETAILS 1 OF 3

PLAN REF. NO. BG28W-08

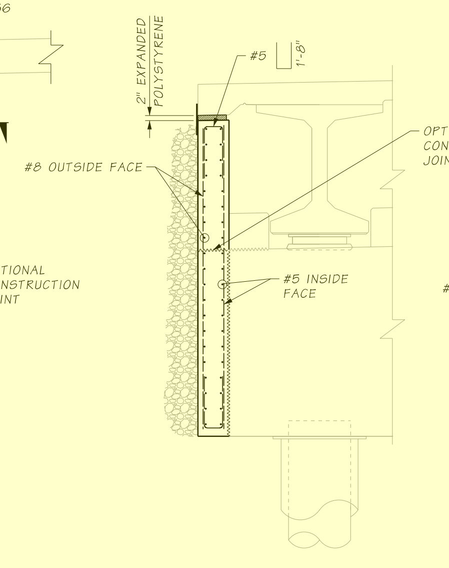
SHEET OF SHEETS



ELEVATION
PIER 2 - NORTH CURTAIN WALL
(LOOKING SOUTH)

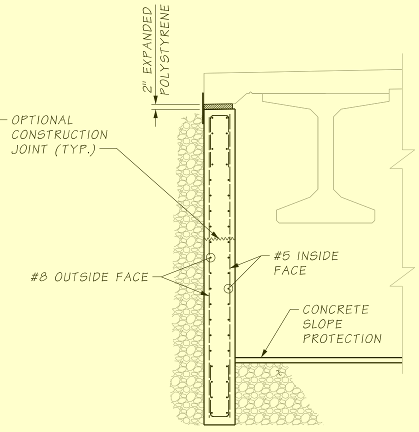


ELEVATION
PIER 2 - SOUTH CURTAIN WALL
(LOOKING NORTH)



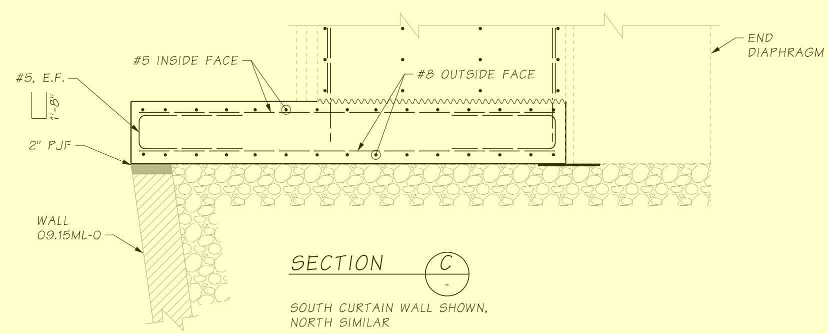
SECTION A

PIER CAP REINFORCEMENT NOT SHOWN. SOUTH CURTAIN WALL SHOWN, NORTH SIMILAR



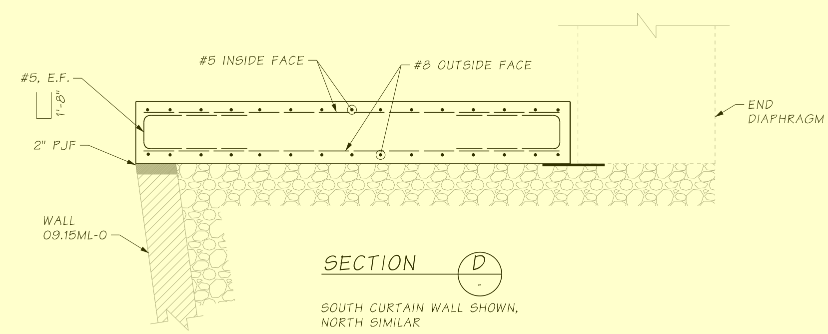
SECTION B

SOUTH CURTAIN WALL SHOWN, NORTH SIMILAR



SECTION C

SOUTH CURTAIN WALL SHOWN, NORTH SIMILAR



SECTION D

SOUTH CURTAIN WALL SHOWN, NORTH SIMILAR

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Design Mgr:	STEVE WOODRUFF	RELEASE FOR CONSTRUCTION RECORD		
Designed By:	M. DASTFAN			
Checked By:	M. BAUGHMAN			
Detailed By:	P. CONRAD			
Current Revision By:				
Date:	4/15/2021	RELEASE FOR CONSTRUCTION	04/16/2021	0
Time:	7:45:29 PM	DESCRIPTION	DATE	NO

REGION NO. 10 | STATE WASH. | FED. AID. PROJ. NO. | SHEET NO. | TOTAL SHEETS
JOB NUMBER XL5467

MATTHEW D. BAUGHMAN, STATE OF WASHINGTON, 49664, STRUCTURAL PROFESSIONAL ENGINEER
DATE
E.O.R. 1 STAMP BOX
DATE
E.O.R. 2 STAMP BOX

Washington State Department of Transportation
FLATIRON LANE
wood. COWI

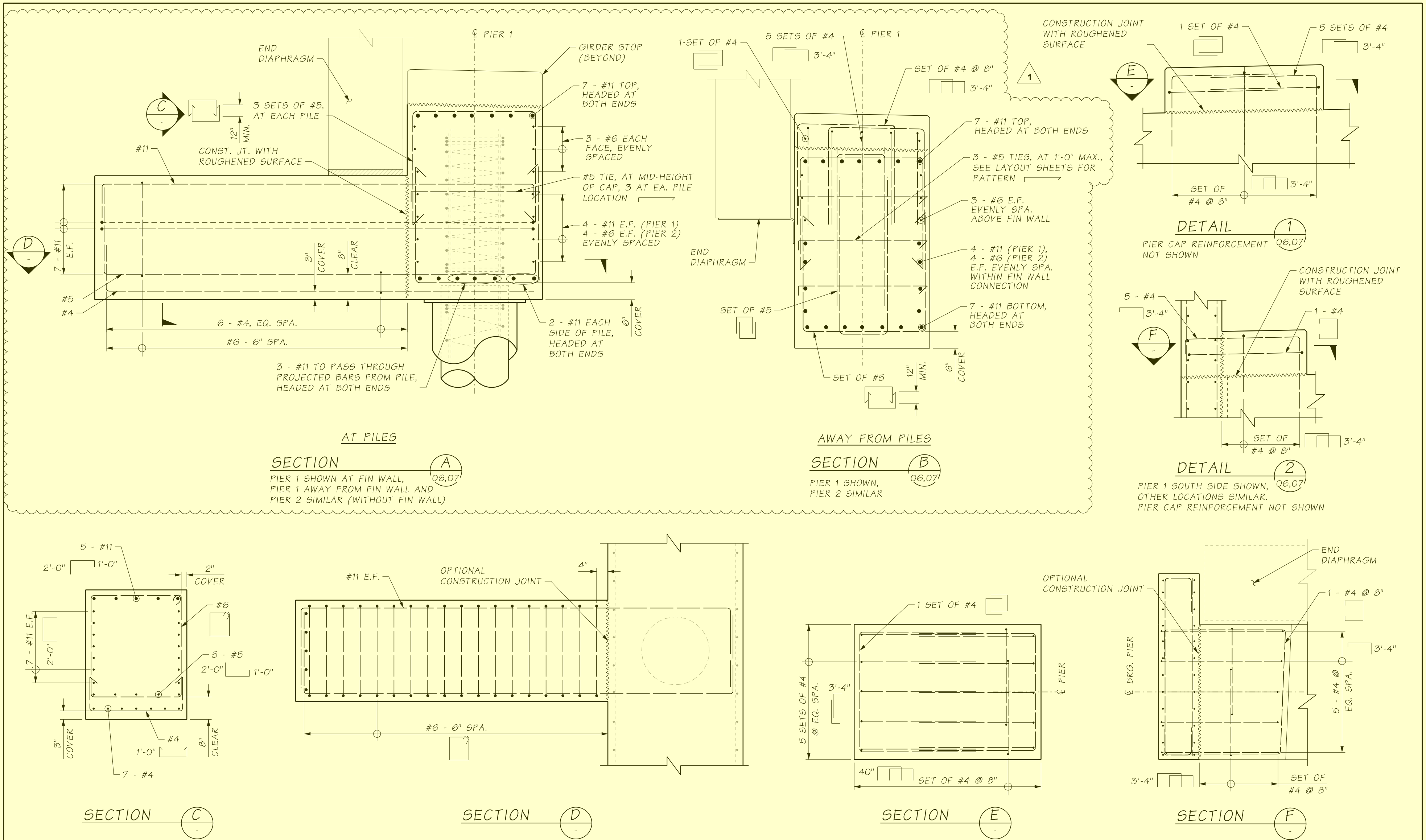
I-405; RENTON TO BELLEVUE WIDENING AND EXPRESS TOLL LANES PROJECT

112TH AVENUE SE OVER SB I-405

PIER DETAILS 2 OF 3

PLAN REF. NO. BG28W-09
SHEET OF SHEETS

I-405 FILE NO. 9242 SHEET



I-405 FILE NO. 9242 SHEET

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Design Mgr:	BRIAN BELL	RELEASE FOR CONSTRUCTION RECORD			
Designed By:	M. DASTFAN				
Checked By:	M. BAUGHMAN				
Detailed By:	P. CONRAD				
Current Revision By:		NDC-XX	09/09/2021	1
Date:	9/14/2021	RFC - BRIDGE 405/28W	06/28/2021	0
Time:	3:35:24 PM	DESCRIPTION	DATE	NO

REGION NO.	STATE	FED. AID. PROJ. NO.	SHEET NO.	TOTAL SHEETS
10	WASH.			

JOB NUMBER XL5467

MATTHEW D. BAUGHMAN STATE OF WASHINGTON 49664 STRUCTURAL ENGINEER PROFESSIONAL ENGINEER

E.O.R. 1 STAMP BOX DATE

E.O.R. 2 STAMP BOX DATE

Washington State Department of Transportation

FLATIRON LANE wood. COWI

I-405; RENTON TO BELLEVUE WIDENING AND EXPRESS TOLL LANES PROJECT

112TH AVENUE SE OVER SB I-405

PIER DETAILS 3 OF 3

PLAN REF. NO. BG28W-10

SHEET OF SHEETS